STRUCTURAL LOAD TESTING OF GEMINI SINGLE JOIST COMPOSITE FLOOR SYSTEM

Alberta
MUNICIPAL AFFAIRS
Innovative Housing Grants Program

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December, 1989

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The views and conclusions expressed and the recommendations made in this report are entirely those of the authors and should not be construed as expressing the opinions of Alberta Municipal Affairs.

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FOREWORD

The project documented in this report received funding under the Innovative Housing Grants Program of Alberta Municipal Affairs. The Innovative Housing Grants Program is intended to encourage and assist housing research and development which will reduce housing costs, improve the quality and performance of dwelling units and subdivisions, or increase the long term viability and competitiveness of Alberta's housing industry.

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TABLE OF CONTENTS

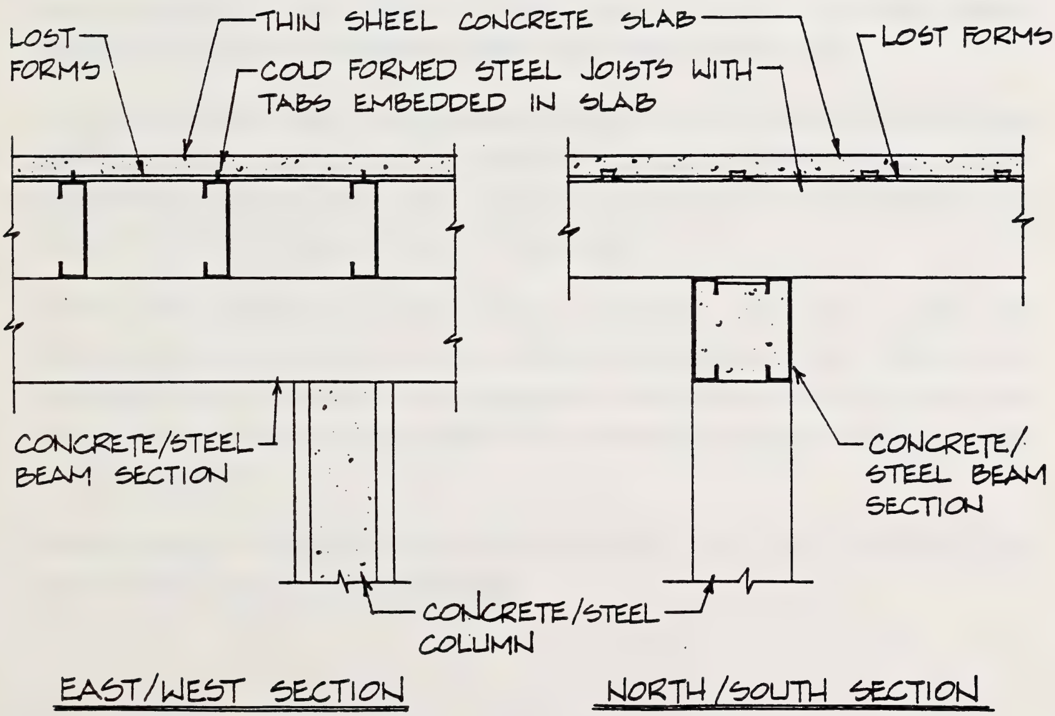
LIST OF FIGURES

LIST OF TABLES

EXECUTIVE SUMMARY

The objectives of this study were to determine the economic viability of the Gemini single joist composite system (Gemini System II) in the residential market, to develop and carry out a testing program for the system, and generate load tables for use by engineers and architects in the design and specification of appropriate structures.

Gemini System II consists of three major components- composite columns, composite beams, and composite floor sections. The term "composite" is used to describe two or more elements, of different materials, acting as a single structural entity. The system is best described by the following diagram.



Formwork for the Gemini System is generally referred to as the "lost form system", since the material stays in place after construction. An earlier successful version of the Gemini System consisted of back-to-back floor joists. The subject of this report (Gemini System II) uses only single joists as shown in the illustration.

A cost study, comparing Gemini System II with conventional methods of construction, was carried out. Gemini System II was 13 % more economical for a seven-storey apartment building than the most economical conventional method of construction, but was 12% more expensive than conventional wood framing for a detached split level house. For single family dwellings, Gemini System II is more suited to a bungalow style and is roughly the same cost as a manufactured composite joist floor system in those applications.

Codes were reviewed and no standardized testing procedure was found for testing the elements of the system components. Therefore, testing procedures were developed for the Gemini System, using previous university testing programs carried out for other composite floor systems as models.

The purpose of the testing program was twofold. - to confirm that the elements of the floor, beam, and column components display composite action, and to provide engineering data from which the structural characteristics and capacities of the components could be calculated.

Testing was carried out in the summer of 1989. Composite action was demonstrated by the components, and precise structural load tables were developed from the test data and subsequent calculations. The load tables will be particularly useful to engineers and architects as design tools for multi-storey structures.

Gemini System II is both versatile and structurally sound, and should find wide application in the construction industry.

1.0 INTRODUCTION

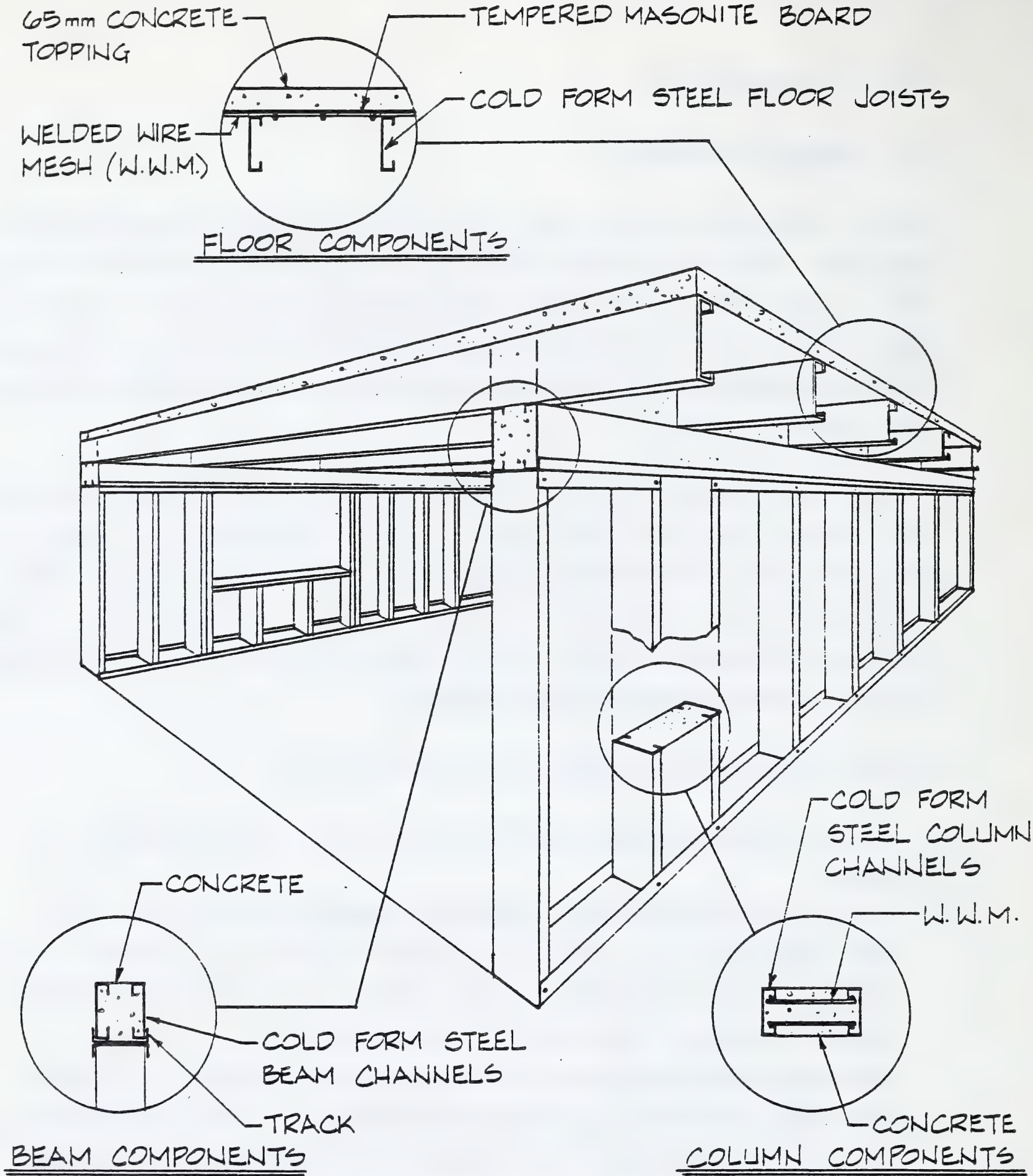
1.1 GEMINI SYSTEM II

Gemini Structural Systems Inc. of Calgary, Alberta has developed a composite concrete thin shell, steel cold-formed joist system which can be used in wall or floor construction. The original Gemini System which has been utilized in the construction of apartments and other multi-use buildings, consists of double (back to back) steel floor joists with a sixty-five millimetre (mm) concrete topping.

The new floor system, Gemini System II, consists of three components- a sixty-five mm concrete floor panel supported by single steel joists, a composite floor beam consisting of concrete filled cold-formed joists, and a composite column. The term "composite" is used to describe two or more elements of different materials acting as a single structural entity. A drawing of the new floor system is shown by Figure 1.1.

The floor system is constructed in the following manner:

- The column channels are attached to the floor below and screwed into place
- The channels required for the beam are installed between columns, with openings at the columns to allow concrete to be placed in the columns. Some intermediate shoring under the beam channels will be required during the installation of the floor joists.
- The cold-formed steel joists are laid in place and screwed to the beam channels. End blocking of the joists is provided, as in wood construction, to prevent the steel joists from twisting. The joists' shear key in the top flange of the joist are bent up vertically.
- 152 x 152 welded wire mesh is laid over the joists as a safety precaution.
- Tempered masonite board (or other suitable forming material) is placed on the wire mesh spanning between tabs on adjacent joists.
- Construction shoring is installed prior to placing of the concrete.
- The concrete is placed in the column and beam channels.
- A fifty mm or sixty-five mm layer of concrete topping is cast to complete the floor system.



Gemini Floor System

Figure 1.1

The Gemini System has several construction advantages:

- it is lightweight and quick to install.
- it can be used with any floor plan configuration.
- it can accommodate several different forming methods and materials.
- it is compatible with other construction systems and can be used in combination with concrete, block, wood and steel structures.
- because of its simplicity, it may not require or will at least reduce the number of field inspections by engineers. This may permit the entry of the Gemini System into the residential construction market as an economical alternative to wood framing, particularly for walk up apartments.

1.2 PURPOSE OF STUDY

Gemini System II represents an evolutionary development of the Gemini System. Where the original Gemini system uses back to back cold-formed steel joists to form an I-beam cross section, Gemini System II, uses only single joists. To gain market acceptance, the evolved system must first undergo thorough structural testing in accordance with accepted engineering practices and principles. The primary purpose of this project was to develop and carry out such a test program. Secondarily this project investigated the economics of the system from a residential perspective, since in addition to structural integrity, the system must prove economically viable to capture a share of that market.

1.3 FORMAT OF REPORT

This report describes in detail the major activities of the project. Section 2 deals with the residential economic viability of the system. Section 3 discusses technical codes and literature which pre-determined many of the requirements for the testing program. Section 4 describes the test procedures and objectives. Section 5 relates to the actual test observations and analyses. Section 6 pertains to the generation of load tables resulting from the engineering analysis of the test data, and Section 7 summarizes the project through discussion of the conclusions reached.

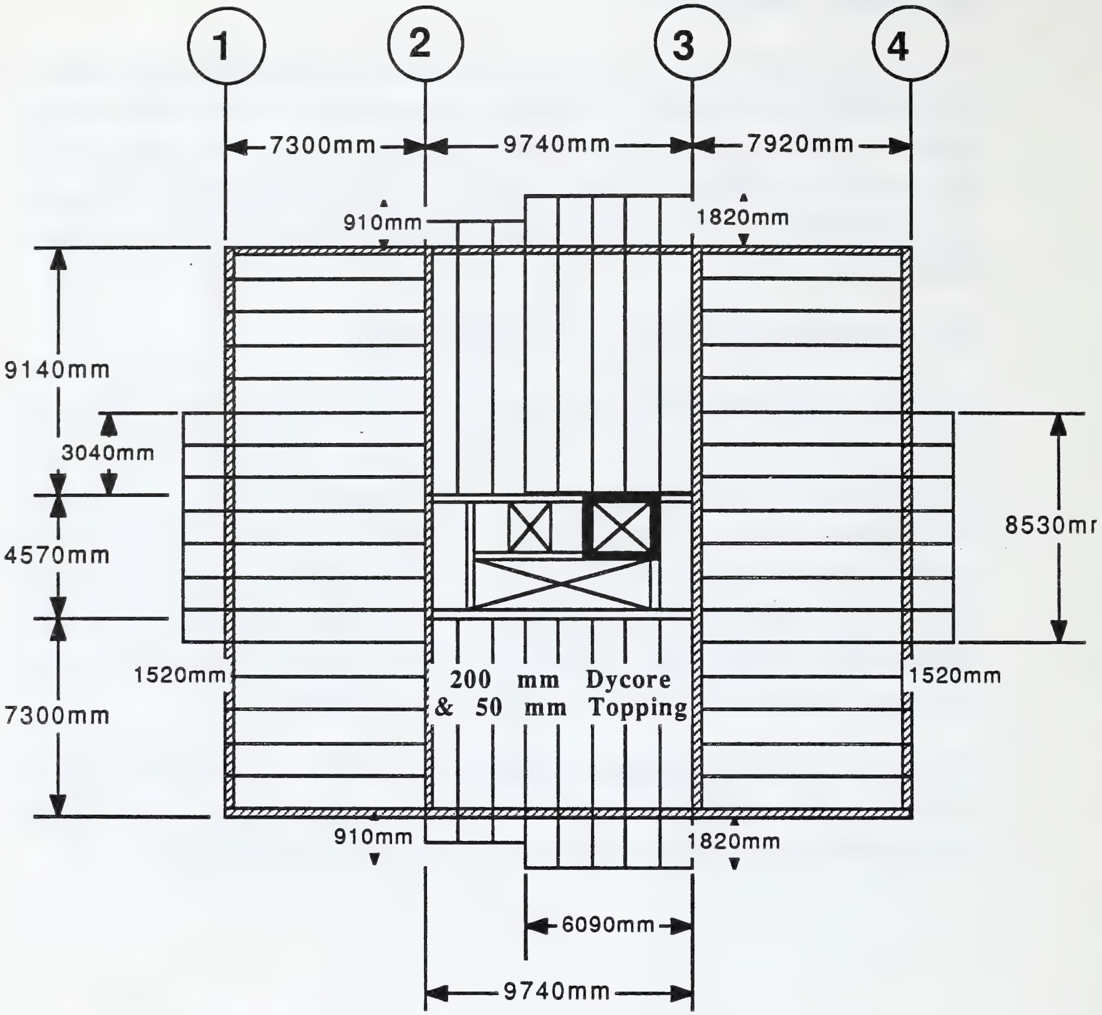
2.0 COST ANALYSIS

The cost of the Gemini II floor system was compared with the costs of conventional construction methods for a typical floor of a seven storey apartment building, and for a single detached house. Cost estimates for conventional construction were supplied by general contractors, while the cost estimate for the Gemini System were provided by H.K. Schilger Enterprises.

2.1 APARTMENT FLOOR SYSTEM COSTS

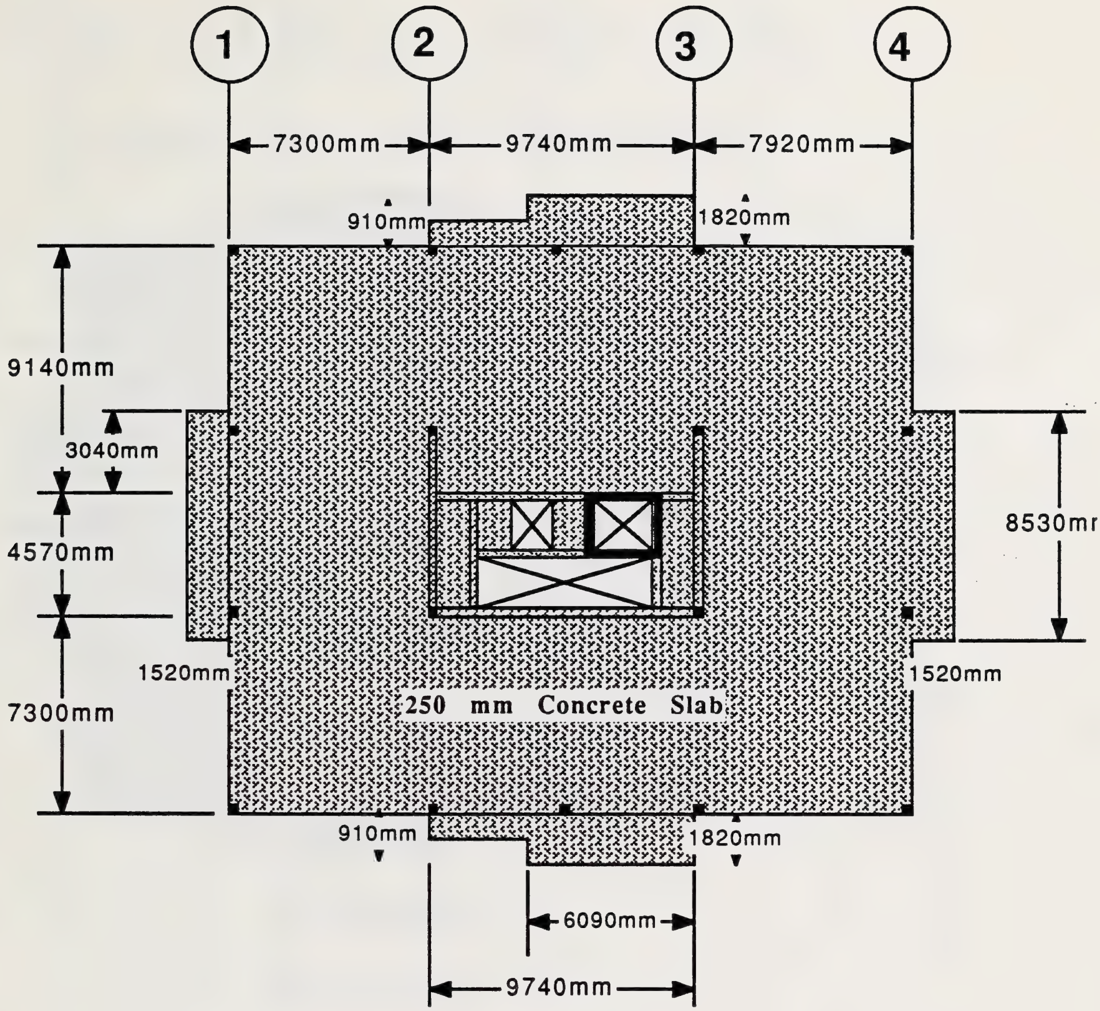
The apartment layout used for cost comparison purposes was a 575 m^2 seven storey precast building actually constructed in Calgary in the 1970's. In addition to the as-built precast design, the building's structural system was redesigned for both a cast-in-place reinforced concrete slab and a bonded post-tensioned concrete slab, using the 1985 Alberta Building Code loading requirements for apartment buildings. Including the Gemini System, these designs yielded four different typical floor systems for cost comparison purposes. Figures 2.1 to 2.4 show the four alternatives.

The cost breakdowns for the four different floor systems are shown in Tables 2.1 through 2.4. As can be seen, the Gemini floor system was 13 % lower in cost than the most economical conventional construction method.



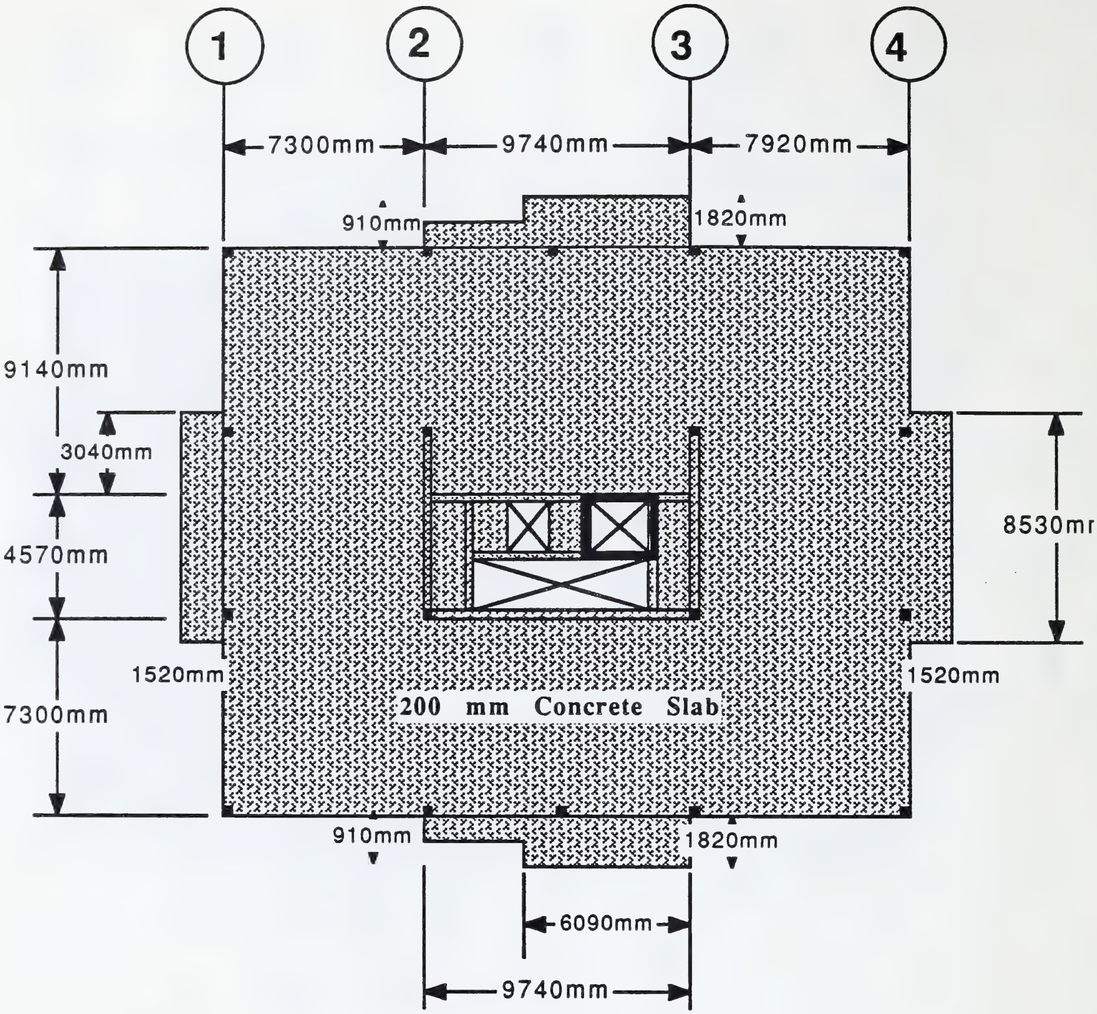
TYPICAL FLOOR
FOR
APARMENT BUILDING
USING
PRECAST CONCRETE CONSTRUCTION

FIGURE 2.1



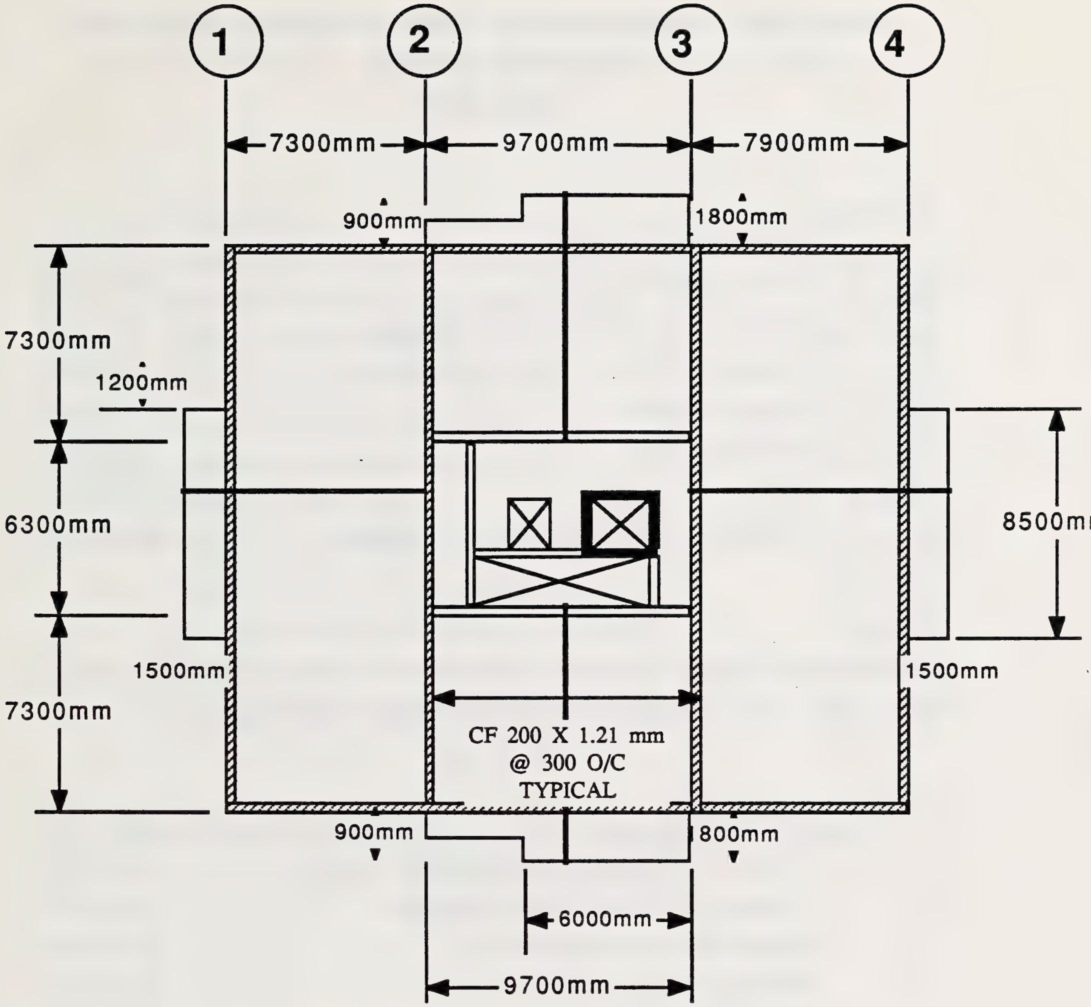
TYPICAL FLOOR
FOR
APARMENT BUILDING
USING
REINFORCED CONCRETE
CONSTRUCTION

FIGURE 2.2



TYPICAL FLOOR
FOR
APARMENT BUILDING
USING
POST-TENSIONED CONCRETE
CONSTRUCTION

FIGURE 2.3



**TYPICAL FLOOR
FOR
APARTMENT BUILDING
USING
GEMINI FLOOR SYSTEM**

FIGURE 2.4

Costs of a Typical Precast Floor in a Seven Storey Apartment Building
Table 2.1

Supply of Hollowcore	$34.0/m^2
Install of Hollowcore	$20.0/m^2
Supply of 50 mm Concrete Topping	$6.0/m^2
Placing of 50 mm Concrete Topping	$5.0/m^2
Supply of Walls	$10.0/m^2
Placing of Walls	$10.0/m^2
Total	$83./m^2
Total Cost without profit and overhead	**$68.6/m^2**

Costs of a Typical Two-Way Mild Reinforced Concrete Floor in a Seven Storey Apartment Building
Table 2.2

Form Work	$30.0/m^2
Supply of 250 mm Concrete Slab	$24.0/m^2
Placing of 250 mm Concrete Slab	$1.0/m^2
Supply of mild reinforcement	$17.5/m^2
Placing of mild reinforcement	$17.5/m^2
Supply of stud walls	$4.0/m^2
Placing of stud walls	$4.0/m^2
Supply of 300 x 300 concrete columns	$2.0/m^2
Placing of 300 x 300 concrete columns	$2.0/m^2
Total	$110.0/m^2
Total Cost without profit and overhead	**$90.9/m^2**

Costs of a Typical Two-Way Post-Tensioned Reinforced Concrete Floor in a Seven Storey Apartment Building
Table 2.3

Form Work	\$30.0/m^2
Supply of 200 mm Concrete Slab	\$22.5/m^2
Placing of 200 mm Concrete Slab	\$1.0/m^2
Supply of post-tensioning	\$11.0/m^2
Placing of post-tensioning	\$11.0/m^2
Supply of mild reinforcement	\$ 3.0/m^2
Placing of mild reinforcement	\$ 3.5/m^2
Supply of stud walls	\$4.0/m^2
Placing of stud walls	\$4.0/m^2
Supply of 300 x 300 concrete columns	\$2.0/m^2
Placing of 300 x 300 concrete columns	\$2.0/m^2
Total	\$102.0/m^2
Total Cost without profit and overhead	**\$84.3/m^2**

Costs of a Typical Gemini Floor System in a Seven Storey Apartment Building
Table 2.4

Shoring and masonite form work	\$12.5/m^2
Supply of 65 mm Concrete Topping	\$6.0/m^2
Placing of 65 mm Concrete Topping	\$5.0/m^2
Supply of 200 mm Steel Cold-form Joists	\$21.5/m^2
Supply of end blocking of joists	\$2.0/m^2
Supply of load bearing stud walls	\$6.0/m^2
Placing of load bearing stud walls	\$6.0/m^2
Total Cost without profit and overhead	**\$59.0/m^2**

2.2 DETACHED HOUSE FLOOR SYSTEM COSTS

The residence chosen for cost comparison work was the 148 m^2 single detached house plan used by Alberta Municipal Affairs for annual cost monitoring. Floor layouts featuring conventional wood framing, truss joist wood framing, and the Gemini System were considered for cost comparison purposes. All the floor layouts meet the requirements of the 1985 Alberta Building Code for residential construction. The Gemini System is not well suited for this type of house since there is schedule incompatibility; specifically, only two of the three floor levels (in the split level house) could be cast at one time; the third would have to be done at a later date. It would, therefore, be recommended that only the lower two levels utilize the Gemini floor system. The floor systems considered are shown in Figures 2.5 to 2.7, with the estimated costs listed in Tables 2.5 through 2.7

Cost for Conventional Wood Framing of the Floors for a Single Detached Home
Table 2.5

Supply of 38 x 235 spruce joists, 10 mm plywood subbase and 16 mm subfloor	\$15.0/m^2
Placing of 38 x 235 spruce joists, 10 mm plywood subbase and 16 mm subfloor	\$15.0/m^2
Supply of 3-38 x 235 D.Fir Beam	0.4/m^2
Placing of 3-38 x 235 D.Fir Beam	0.4/m^2
Supply of two concrete footings and steel posts	1.7/m^2
Placing of concrete footings and steel posts	1.7/m^2
Total	34.2/m^2
Total without profit and overhead	**\$28.2/m^2**

Cost for Manufactured Composite Joist Wood Framing of the Floors for a Single Detached Home

Table 2.6

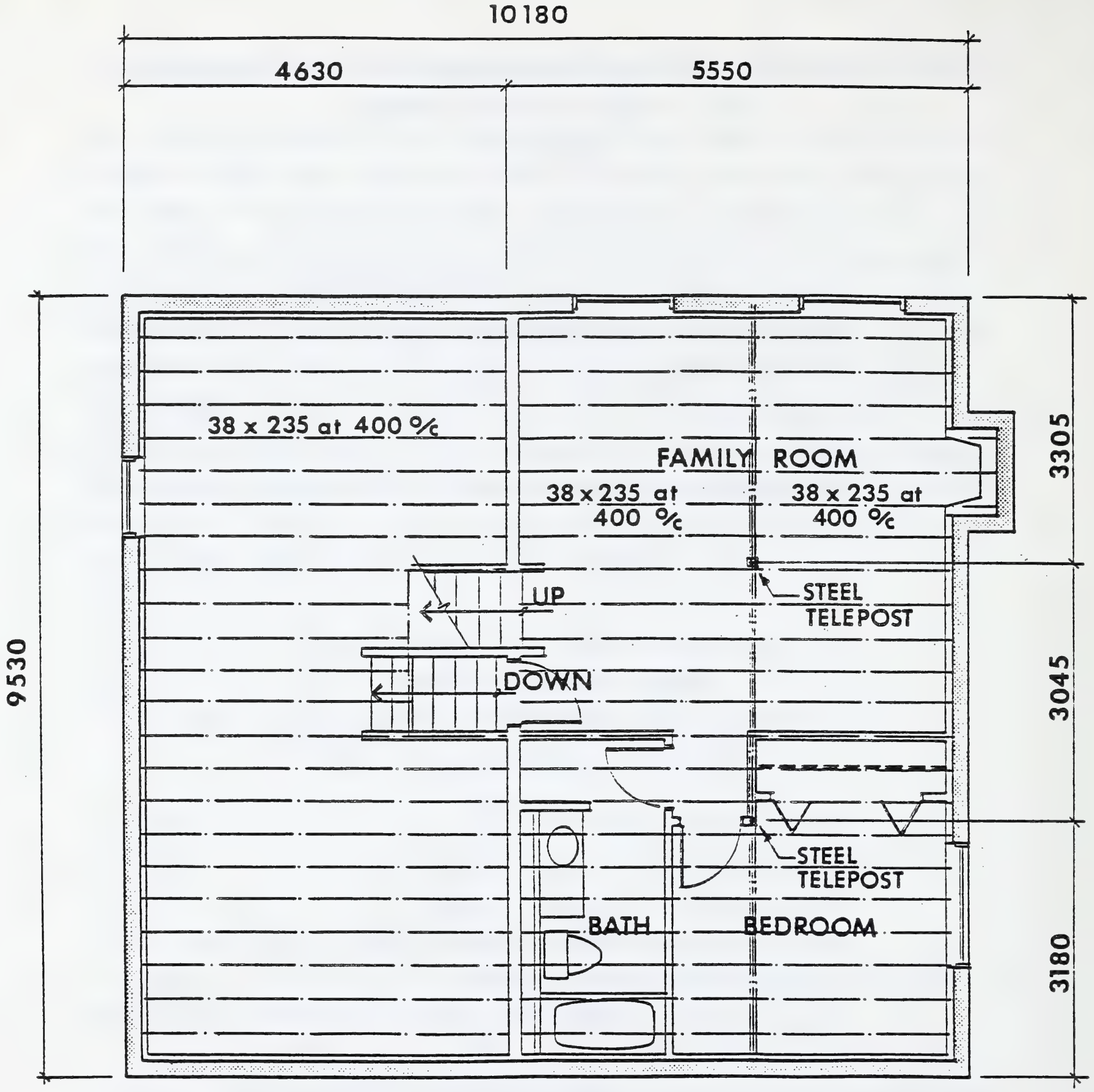
Supply plywood subbase and 16 mm subfloor	$9.0/m^2
Placing plywood subbase and 16 mm subfloor	$9.0/m^2
Supply of 235 TJI	$10.0/m^2
Placing of 235 TJI	$10.0/m^2
Total	38.0/m^2
Total without profit and overhead	**$31.4/m^2**

Cost of Gemini Floor System for a Single Detached Home

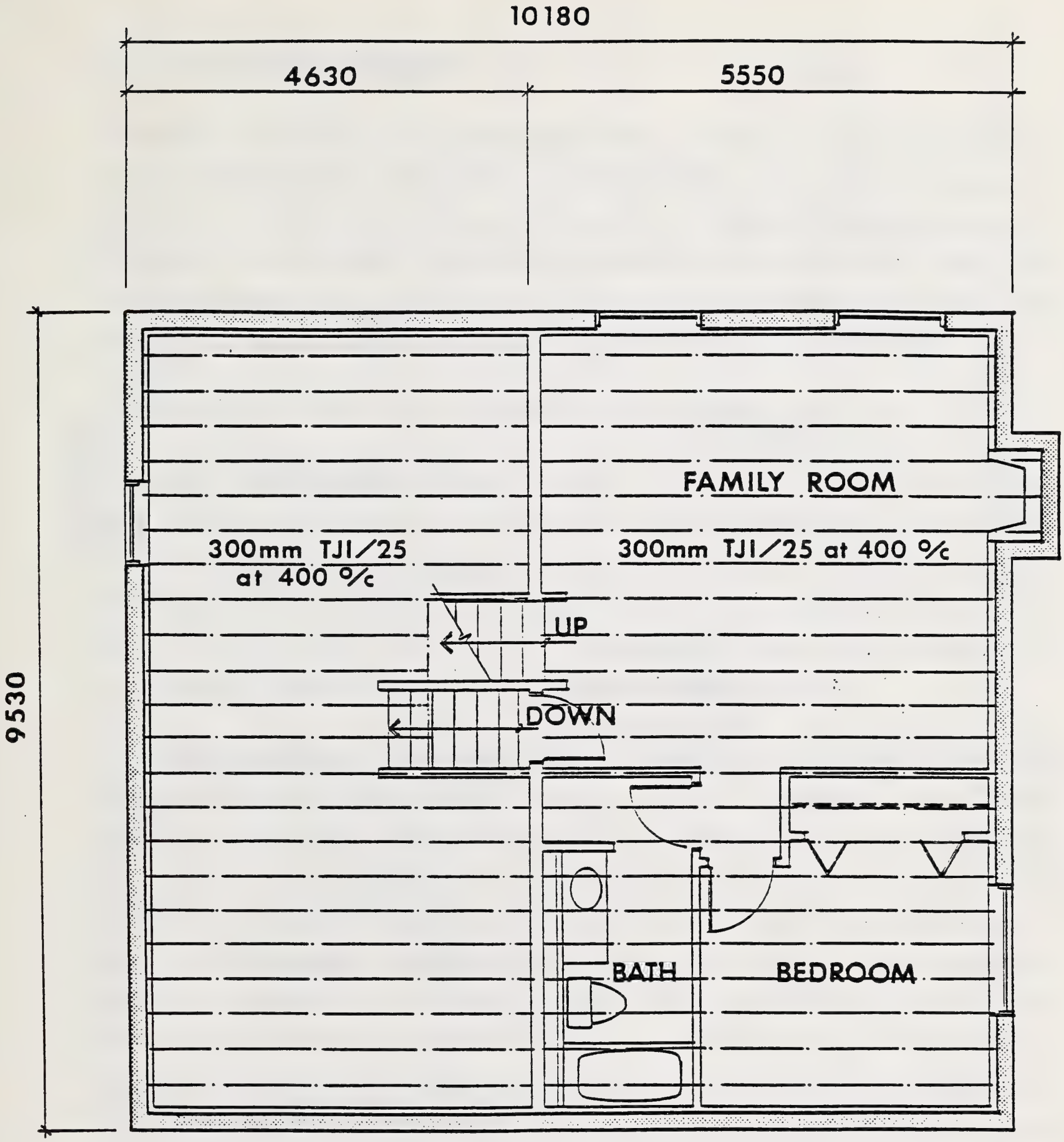
Table 2.7

Shoring and masonite form work	$6.0/m^2
Supply of 50 mm Concrete Topping	$4.5/m^2
Placing of 50 mm Concrete Topping	$4.5/m^2
Supply of 185 mm Steel Cold-form Joists	$16.1/m^2
Supply of end blocking of joists	$0.6/m^2
Total Cost without profit and overhead	**$31.7/m^2**

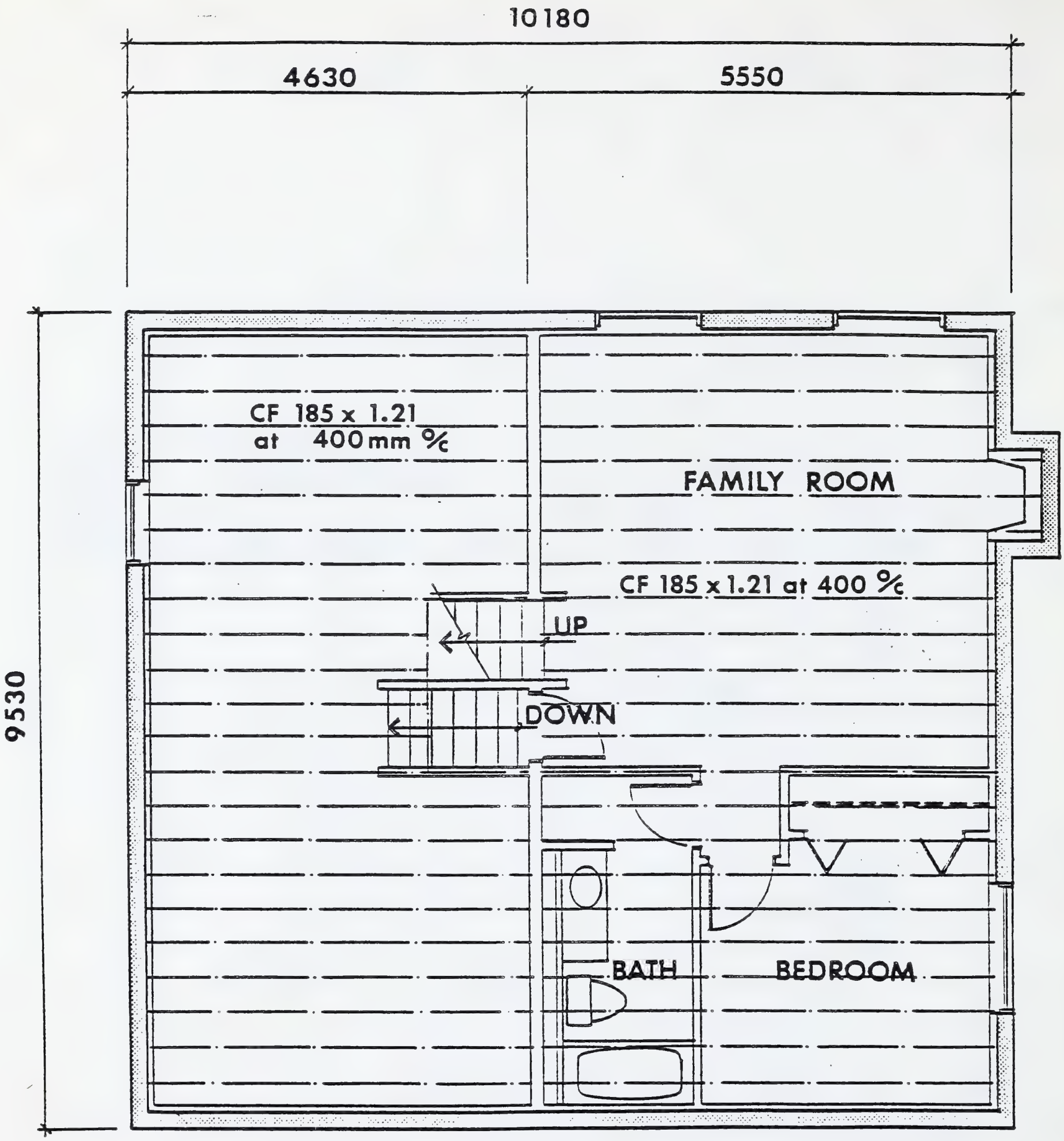
Although the Gemini System is more expensive than dimensioned lumber, it is roughly the same cost as a manufactured composite joist system.



Conventional Wood Framing of the Floors
for a Single Detached Home
Figure 2.5



Manufactured Composite Joist Wood Framing of the Floors
for a Single Detached Home
Figure 2.6



Gemini Floor System
for a Single Detached Home
Figure 2.7

3.0 LITERATURE REVIEW

Composite construction of concrete and steel members is commonly thought to be a relatively new construction method, in fact, composite construction has been in use for some time. This section reviews the development of composite concrete-steel construction research for the past fifty years, as well as reviewing the present design codes, and university experiments that have helped in the development of the study's experimental program.

3.1 BRIEF HISTORY OF COMPOSITE DESIGN

Composite steel beam concrete design was first included in the 1942 National Building Code of Canada and was subsequently used in the American bridge code in 1944. Two years later the American Institute of Steel Construction (AISC) developed provisions on composite design to be used in the 1948 AISC design code. The design of beams was based on a linear elastic stress distribution through the composite section.

In the 1960's further research was done to investigate the use of an ultimate capacity of the composite beam section. In 1964 Chapman proved that composite beams could achieve an ultimate capacity using a plastic section of the steel beam, providing there was enough shear connection to take the maximum horizontal force at failure. In 1965 Sutter and Driscoll, based on 9 shear stud push-out specimens, 12 composite beams, and previous research developed the moment capacity model used today. Sutter and Driscoll concluded that, provided there is enough shear connection between the zero and maximum moment to resist the horizontal force of the yielded beam section, full moment capacity can be achieved. They further concluded that slip between the concrete and steel can be tolerated for full moment capacity, provided the shear connectors have sufficient longitudinal shear capacity.

3.2 GOVERNING STANDARDS

Although the performance of the Gemini II composite system was verified by an experimental program, the design methodology must also be shown to conform to accepted engineering practice. Therefore, both testing and design calculation must reflect the philosophy of the appropriate design

codes permitted in the Alberta Building Code. The design codes reviewed for this study were:

- CAN3-A23.3-M84 Design of Concrete Structures for Buildings
- CAN3-S136-M84 Cold-formed Steel Structural Members
- CAN3-S16.1-M84 Steel Structures for Buildings Limit States Design

3.2.1 Design of Concrete Structures for Buildings

CAN3-A23.3-M84, **Design of Concrete Structures for Buildings** is the governing design code for concrete members in Canada. Although the code does not provide any guidance in the design of composite concrete steel members, it does provide a testing method for beams and joists. Therefore, the loading sequence used in the experiments should satisfy the criteria required in section 20 of A23.3.

3.2.2 Cold-Formed Steel Structural Members

CAN3-S136-M84, **Cold-formed Steel Structural Members** is the governing design code for cold-formed steel members in Canada. Although the cold-formed steel members' strength is governed under S136, the design code does not address the design of composite concrete and structural steel members. Therefore, this system cannot be designed using S136 exclusively. S136 also does not provide any guidance in testing the system.

3.2.3 Steel Structures for Buildings - Limit States Design

CAN3-S16.1-M84, **Steel Structures for Buildings - Limit States Design,** is the governing design code for hot rolled steel members in Canada. The code cannot be used in the design of cold-formed members; however S16.1 has had a composite design section for beams since 1974, including a method for calculating the degree of composite action of the steel beam with the concrete topping, and a model to use in calculating the moment capacity. Unfortunately, these methods were based on experiments using compact, and symmetrical (Class 1 & 2 Section) beams. To date, the Canadian Institute of Steel Construction does not know of any experimental work which has been done using beams that will fail due to elastic lateral torsional buckling (Class 4 Section), i.e. a cold-formed section. But, S16.1

can be used as a starting point for the development of a design model for the composite system.

3.3 UNIVERSITY EXPERIMENTAL PROGRAMS

3.3.1 Shrinkage and Flexural Tests of Two Full-Scale Composite Trusses

"Shrinkage and Flexural Tests of Two Full-Scale Composite Trusses" by Anita Bratland and Dr. D.L. Kennedy at University of Alberta, 1986, provided the basis for the testing arrangement used for both the Gemini floor joists and floor beams. In this masters thesis, the authors describe a four point loading setup that accurately simulates uniform loads on beams.

3.3.2 Behaviour of Composite Concrete Column with Cold-Formed Steel

"Behaviour of Composite Concrete Column with Cold-Formed Steel" by Kwok-Cheung Chung at the University of Windsor , 1986, describes an experimental program for developing column design interaction diagrams. Similar techniques were employed in the testing and analysis of the Gemini System II column components.

Using the information gathered in the literature research an experimental program was developed. This program is discussed in the next section.

3.4 JURISDICTIONAL AUTHORITY

Authority for approval to use the Gemini System for construction projects in Alberta rests with the Government of Alberta, Department of Labour, Building Standards Branch. In determining the status of approval of a new or revised structural system that Branch relies extensively on the expertise and integrity of the engineering firm or individual involved in designing and/or testing the system. Engineering calculations and results, to which the seal of the responsible engineer has been affixed, are submitted to the Building Standards Branch for review and consideration, and approval or rejection of the system is accordingly determined.

4.0 TEST PROGRAM

Testing investigated the two components of the members- the concrete topping, and the cold-formed steel - and the connection between the cold-formed steel and concrete. Concrete was evaluated using compressive strength tests, and steel properties were found using tensile coupon tests. The testing program also included two full scale tests of the joists, and beams, and three for the columns. A flow diagram of the overall experimental program is shown in Figure 4.1

4.1 CRITERIA GOVERNING THE EXPERIMENTAL PROGRAM

The objectives of the experimental program were twofold:

.1 to evaluate the behaviour of the members, specifically observing the connection between the concrete and the steel to ensure composite action.

.2 To validate the engineering models that would be used in conjunction with existing design codes, CAN3-S136 and CAN3-A23.3, to generate structural load tables.

Present structural engineering design requires both the ultimate capacity (breaking strength) and the service capacity (working strength) of the members be calculated. Further, to facilitate the generation of load tables, it was necessary to be able to express ultimate capacity in terms of shear capacity of the member, moment capacity of the member and, for columns, axial capacity. By the same token, the serviceablilty capacity was deemed to be limited by the maximum loads that would cause appropriate deflections, such as L/240 for total loads, and L/360 for live loads, and by limiting the natural frequency of the system to greater than 4Hz. The testing program was therefore structured such that these parameters could be measured or resultantly calculated.

4.2 INDIVIDUAL EXPERIMENTS

The experimental program consisted of seven full scale tests, eighteen push out tests, and auxiliary tests. These tests are described in the following subsections.

4.2.1 Floor Beam and Joist Tests

Two full scale tests were carried out for both the beam and joist components. The beams and joists were tested in a similar manner (see Figure 4.2), but different configurations were used.

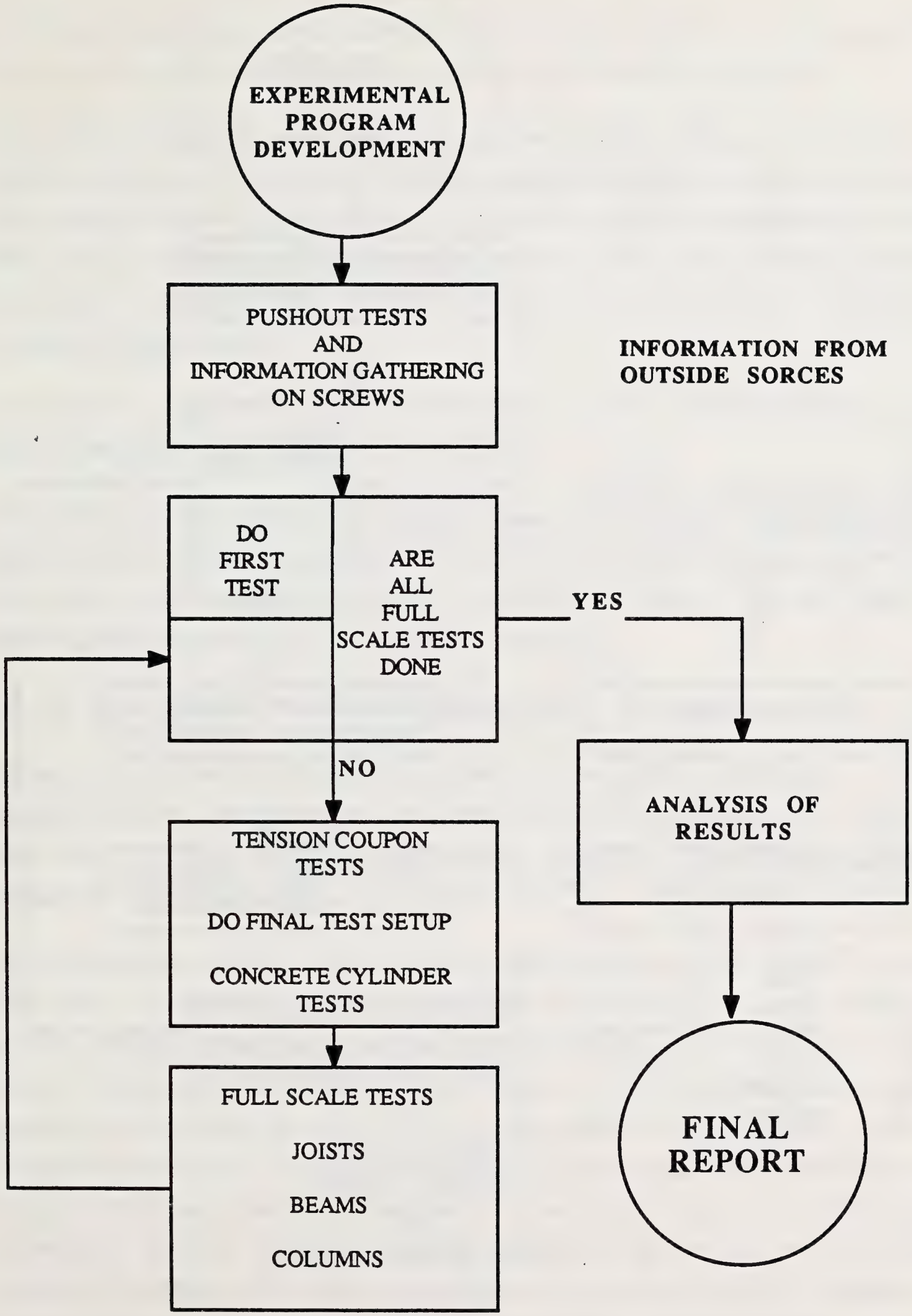
4.2.1.1 Beam and Joist Test Specimens

The four full scale test specimens (two beams, and two floor panels) are shown in Figures 4.3 through 4.10. The specimens comprised a composite system consisting of 18 gauge (0.91 mm) and 20 gauge (0.76 mm) steel joists with shear connectors (Figure 4.6), and concrete topping. There is a 9 mm space between the top of the joist and the concrete topping for form work. All the test specimens were fabricated at Rocky Mountain Precast, Calgary.

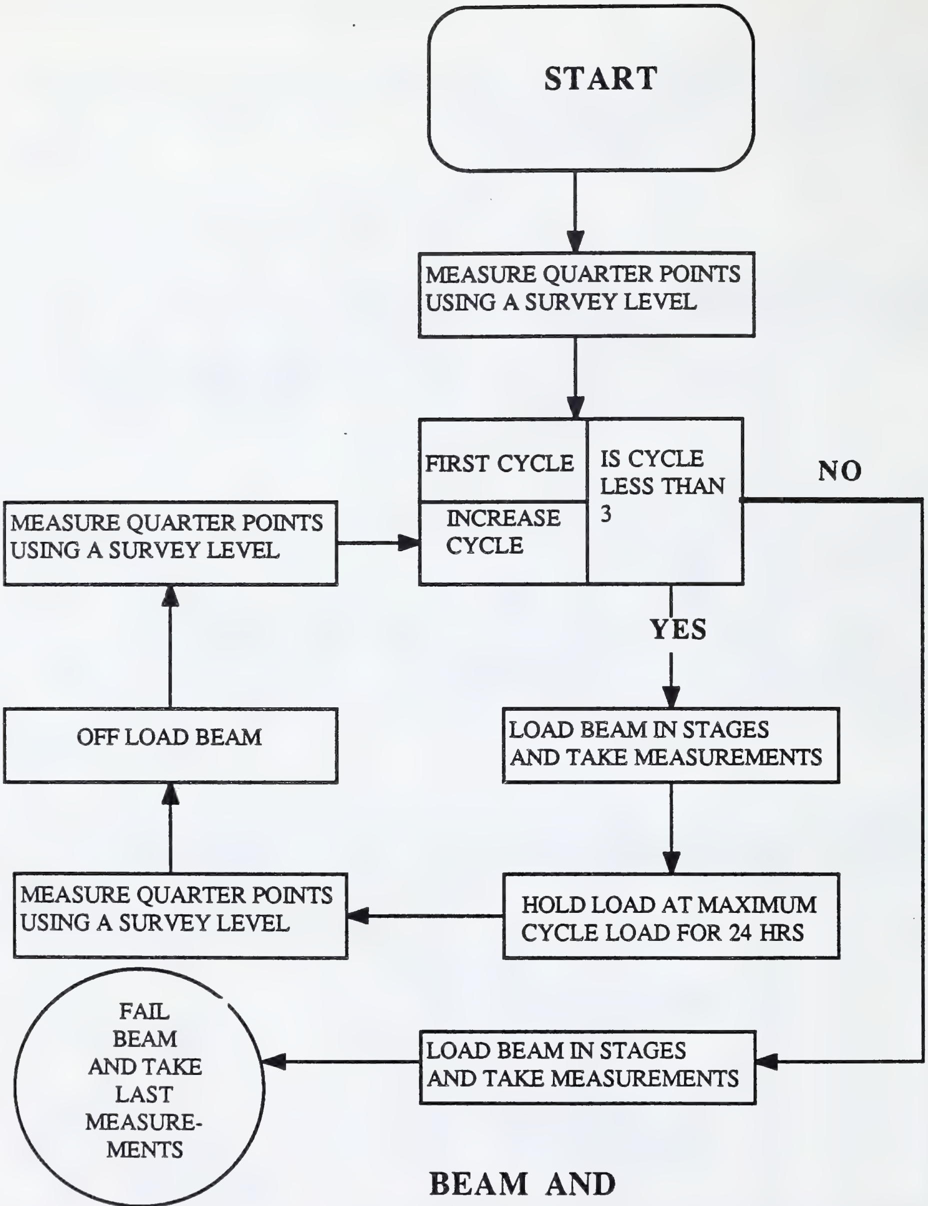
The concrete used in the first beam and floor panel was not a residential mix design but a precast panel mix. The mix had a 28 day design strength of 38 MPa. The concrete mix design used was:

Cement (type 30)	415 kg per cubic metre
Water	175 kg per cubic metre
Aggregate	915 kg per cubic metre
Air	4 - 6%
Unit weight	2200 kg per cubic metre
7 day test strength	26.3 MPa (100 x 200 cylinder)

It should be noted that the concrete was placed in the form without vibration to simulate field conditions. The specimens were tested with an estimated concrete strength of 23 MPa. The first floor beam was retested later with the second floor panel. The second floor panel was cast using residential concrete.



EXPERIMENTAL PROGRAM
FIGURE 4.1



BEAM AND JOIST LOAD TEST FLOW DIAGRAM

FIGURE 4.2

Two field cured concrete test cylinders were broken at the time of testing and the average concrete strength was 23 MPa for the second floor panel.

The cold-formed joists were assumed to have a yield strength of 230 MPa (33 ksi) in the first floor panel, and the steel was tested after the load test. The tension tests indicated that the steel had a yield strength between 310 MPa and 329 MPa, with an ultimate strength of 361 MPa. These values were used in the evaluation of the first floor panel results and in setting up the later full scale tests.

It should be noted that the first floor beam had four additional struts to prevent the beam from overturning.

The last beam test was fabricated as shown in Figure 4.10 and was fully instrumented with strain gauges. The specimen was fabricated as a mini floor system to evaluate the method of load transfer between the floor joists, as well as the behaviour of the two beam channels.

4.2.1.2 Floor Beam and Joist Experimental Procedure

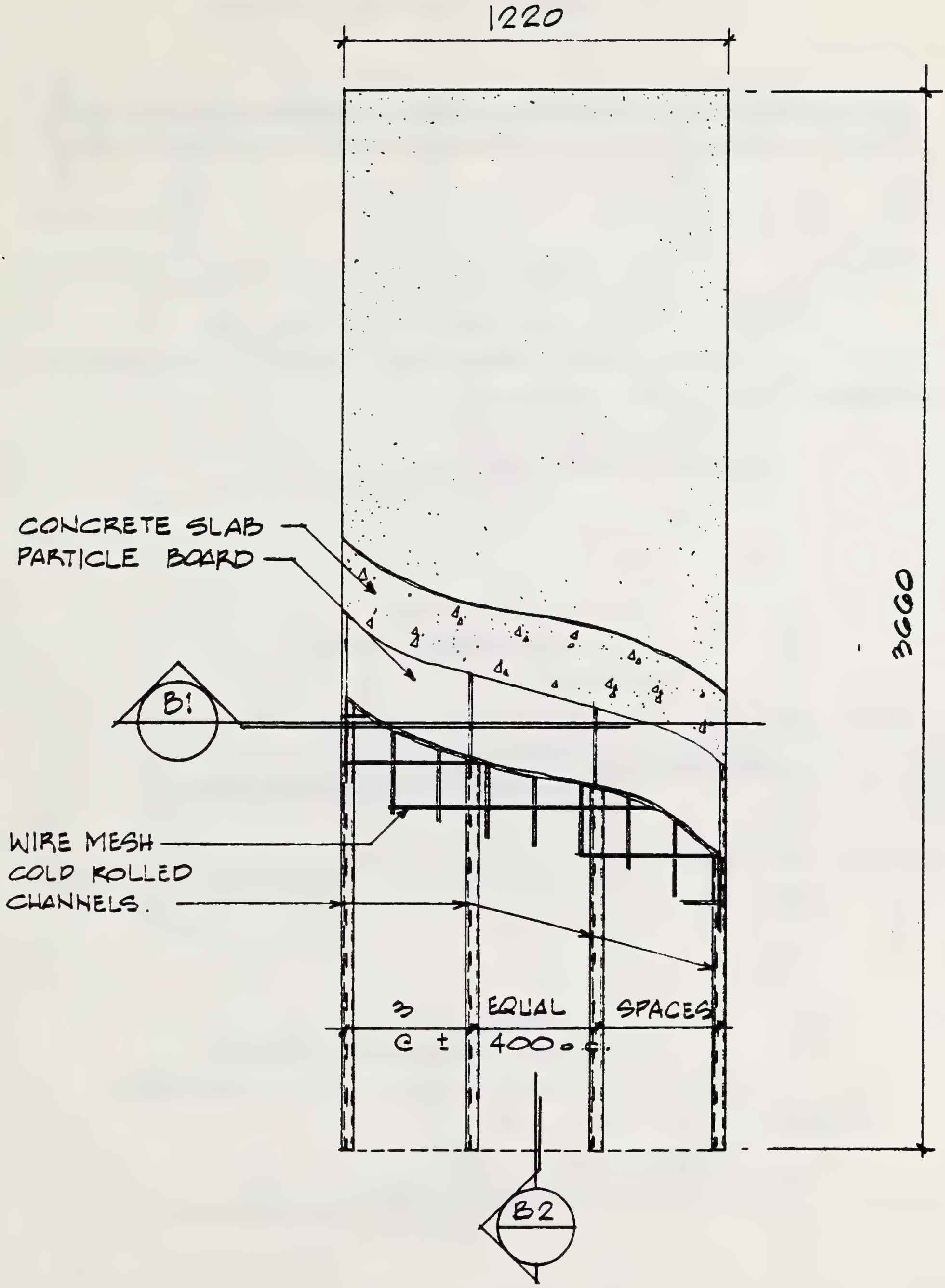
Figure 4.11 shows the test setup used for the beams and joists. The bending moment induced by the test closely approximated that of an uniform load while the shear diagram is noticeably different. This difference was accounted for in the analysis.

All the specimens, except beam panel II, were tested in three cycles. In the first cycle the specimens were loaded in increments up to the design working stress capacity as specified by CAN3-S136-1974, except for the second floor panel where the load was taken up to 90 % of the yield stress as permitted by CAN3-S-16.1. The load was held on the specimens for 36 hours, then removed. The deflection rebound was measured to determine if the specimens were behaving elastically. The second cycle continued with incremental loading up to the specified yield strength of the cold-formed member (230 MPa). This load was held for a 48 hour period, then released. In the final cycle the specimens were loaded until failure occurred. Floor beam panel II was loaded in one cycle to failure since the previous tests

proved that the shear connections between the concrete and steel channels would not loosen under cyclic loading.

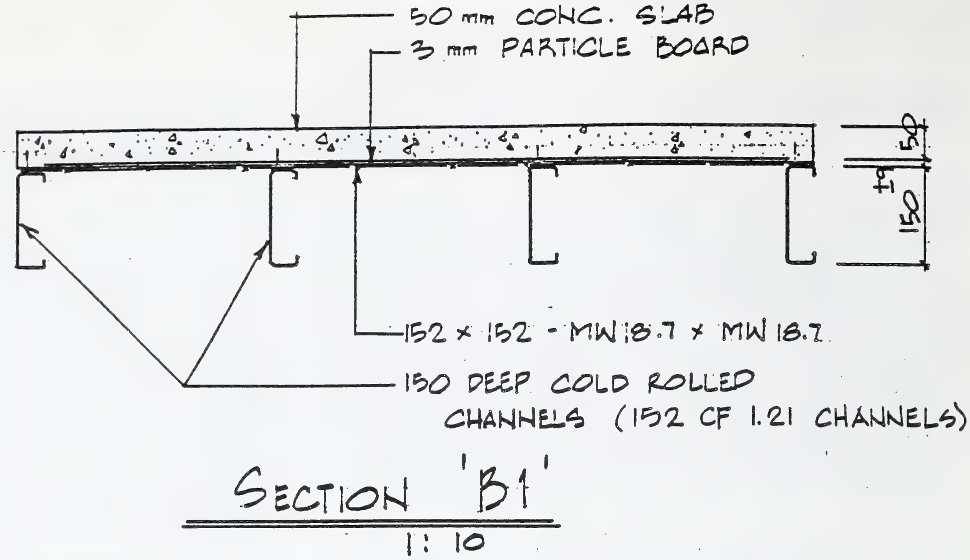
Deflections were measured after each load increment, using a different methodology for the beam than for the floor panel. Floor panel deflections were measured at six points (1/4 points and mid-span on both sides) using tapes and a wire pulled tight from support to support. The incremental deflections were measured on these tapes. These deflections were checked by measuring the elevations of the points at the beginning and end of each cycle using a surveying level. No differences were found between the two methods of measurement.

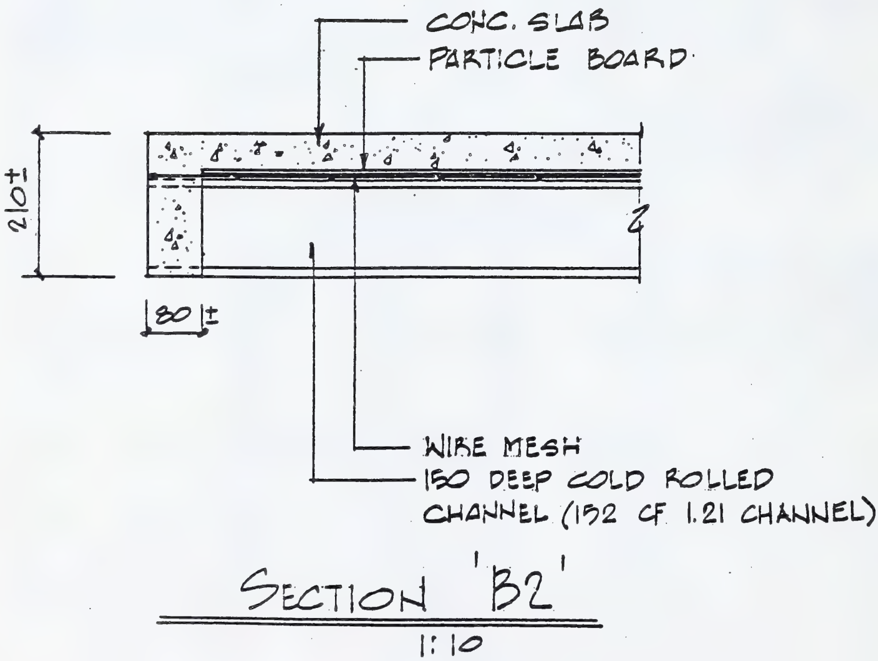
Only the midspan deflection was measured on the beam; this was done using a surveying level, taking the elevations at the ends of the beam and at midspan, to give the relative midspan deflection. The loads were applied using either weighed bags of aggregate or coils of sheet metal.



Floor Joist Panel I

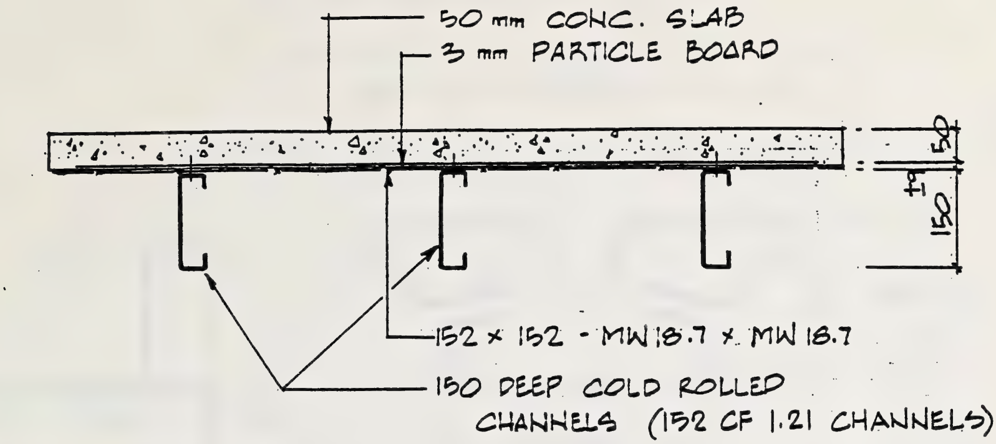
Figure 4.3





Floor Joist Panel I

Figure 4.4



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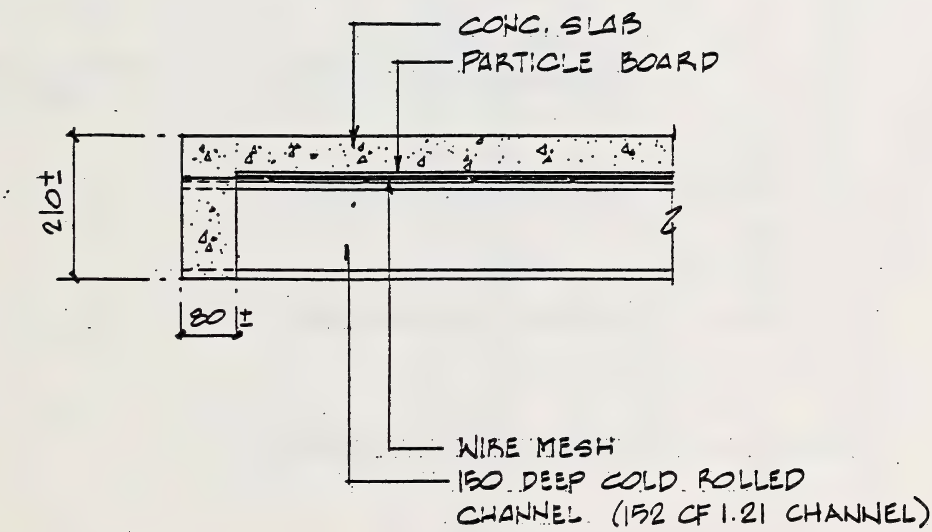
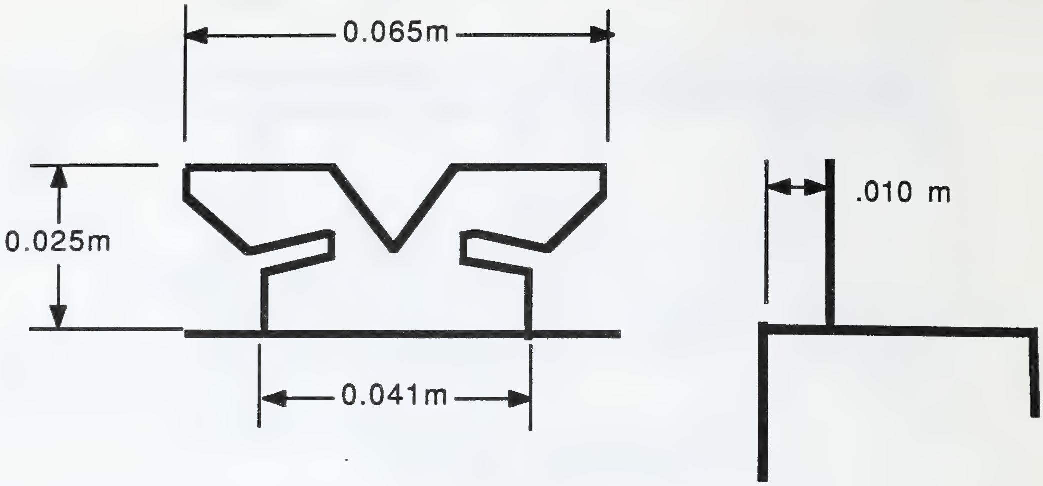


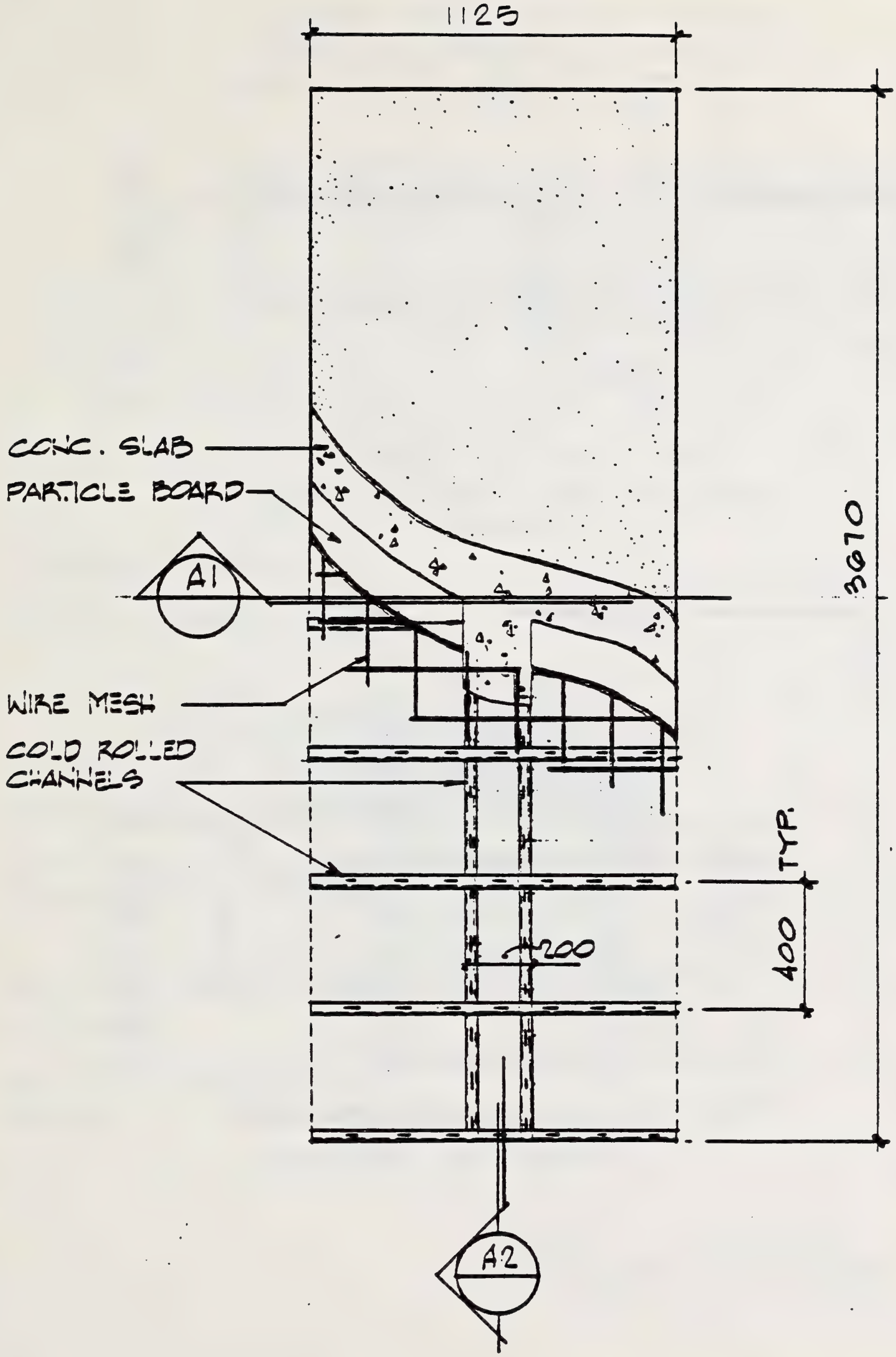
SECTION 'B2'
1: 10

Floor Joist Panel II

Figure 4.5

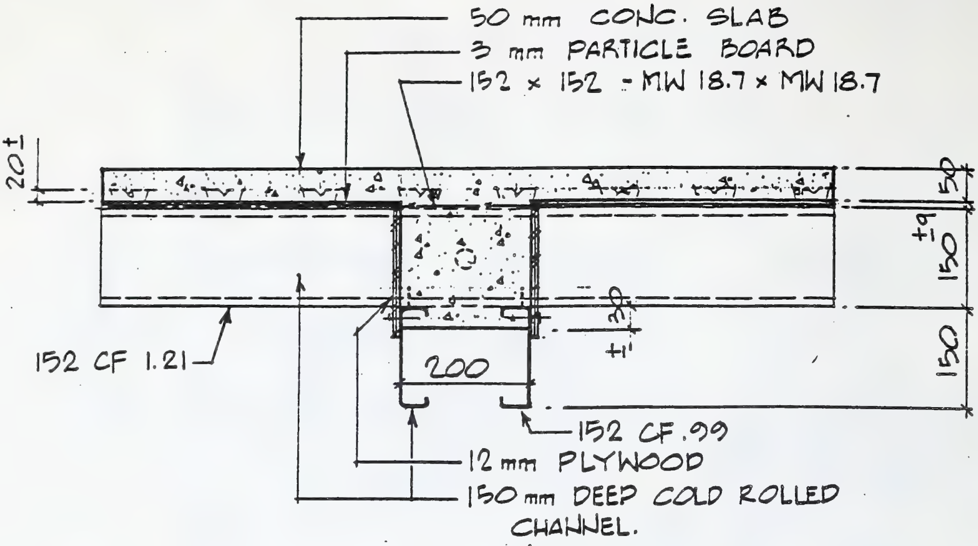


Shear Tab
Figure 4.6



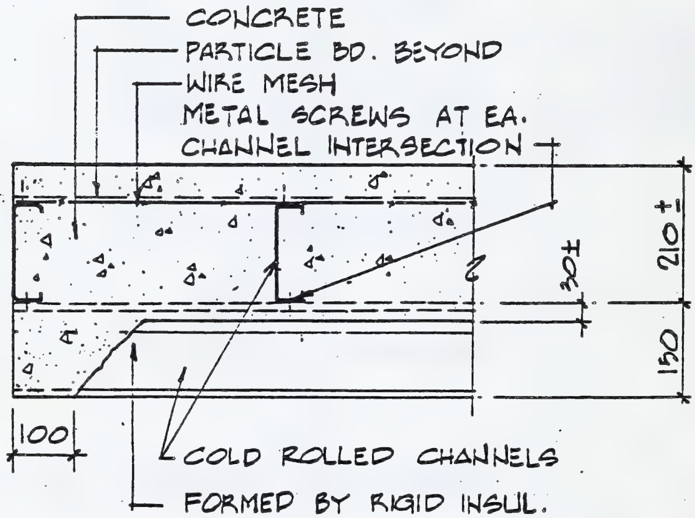
Floor Beam Panel I

Figure 4.7



SECTION 'A1'

1:10

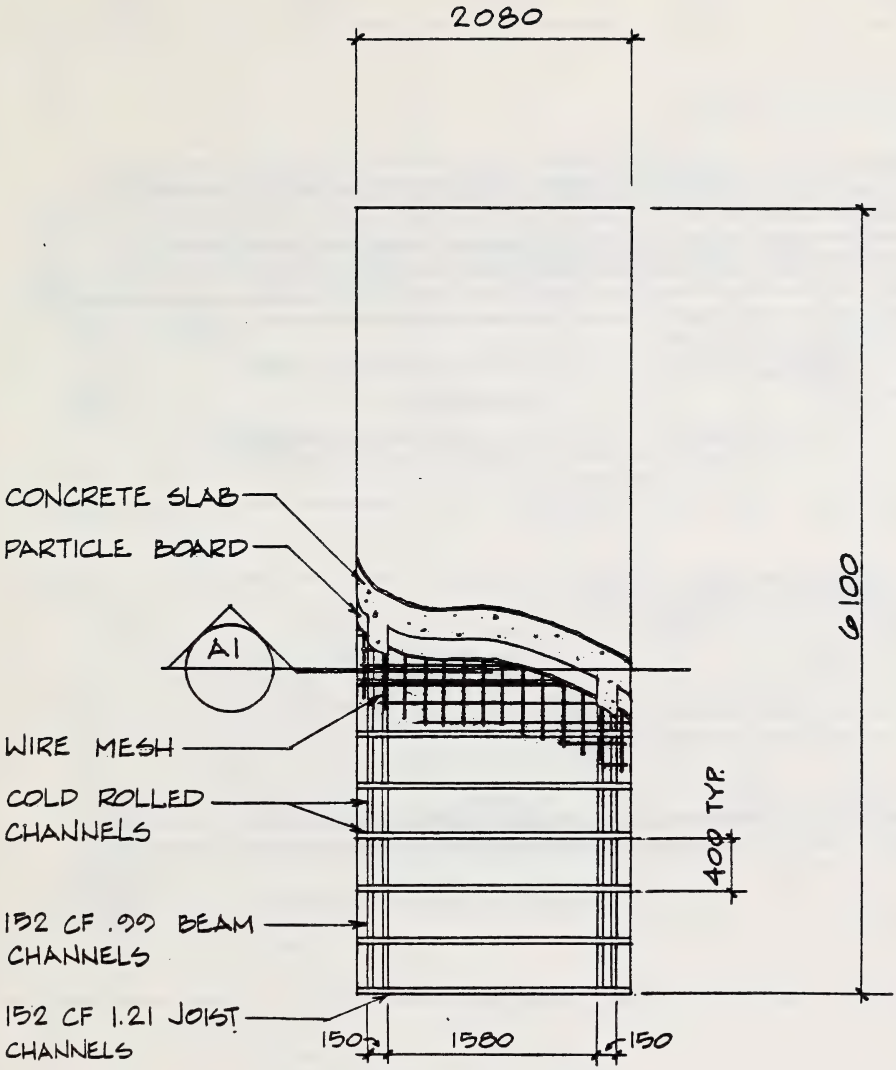


SECTION 'A2'

1:10

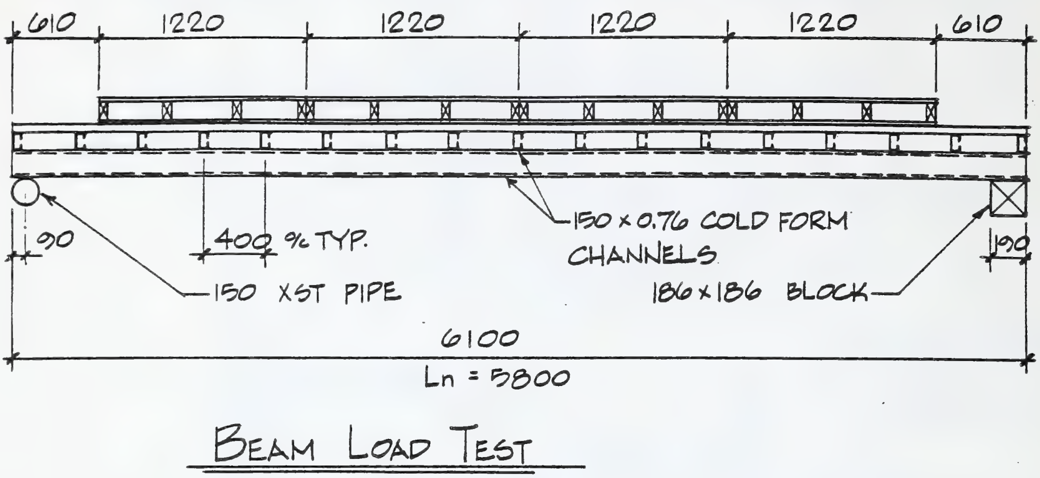
Floor Beam Panel I

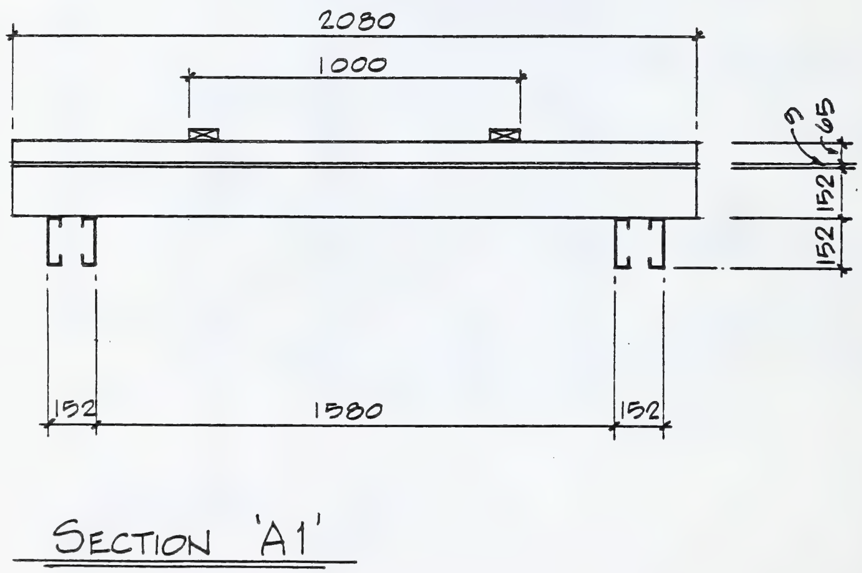
Figure 4.8



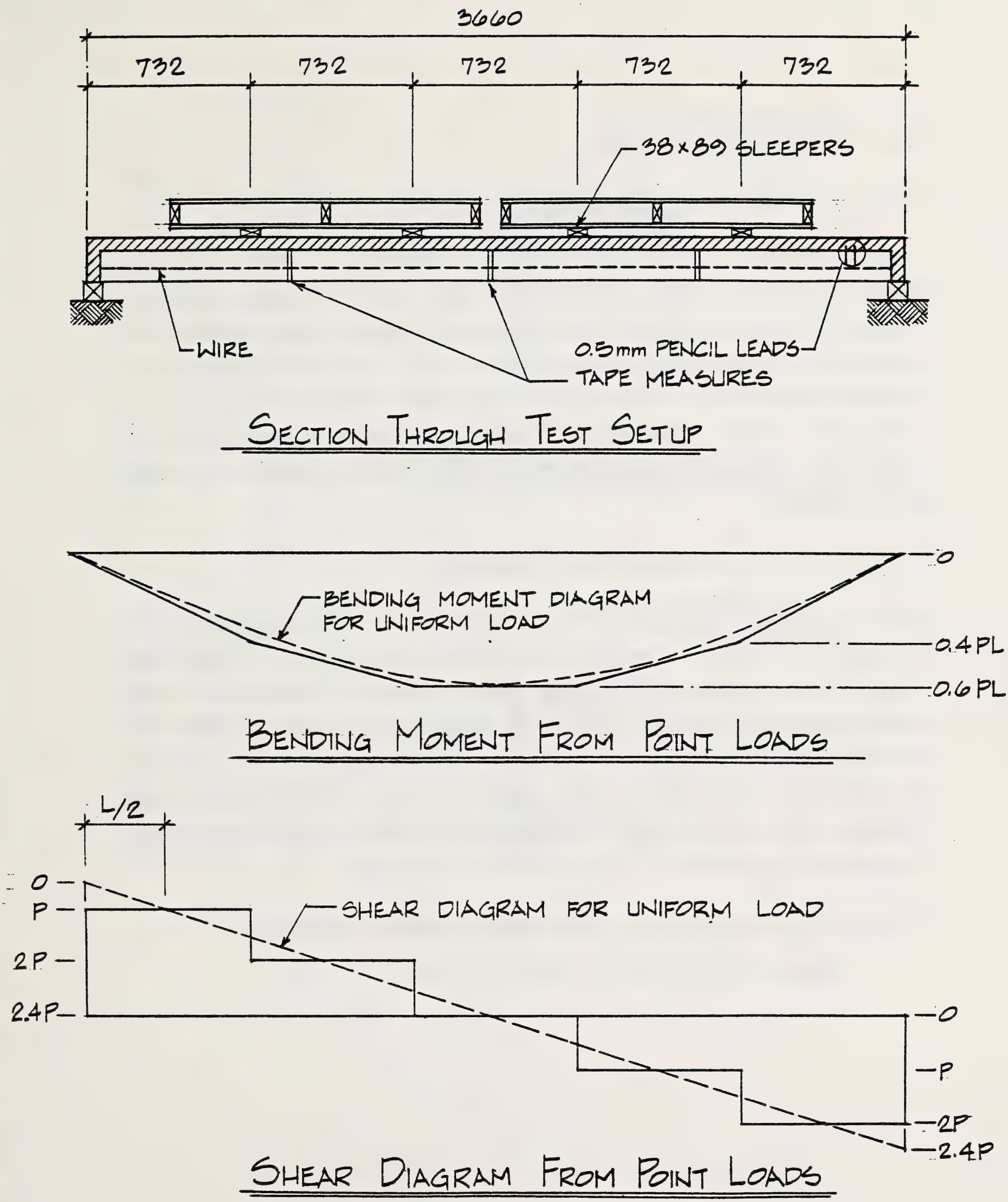
Floor Beam Panel II

Figure 4.9





Floor Beam Panel II
Figure 4.10



Floor Joist and Beam Test Setup

Figure 4.11

4.2.2 Column Experiments

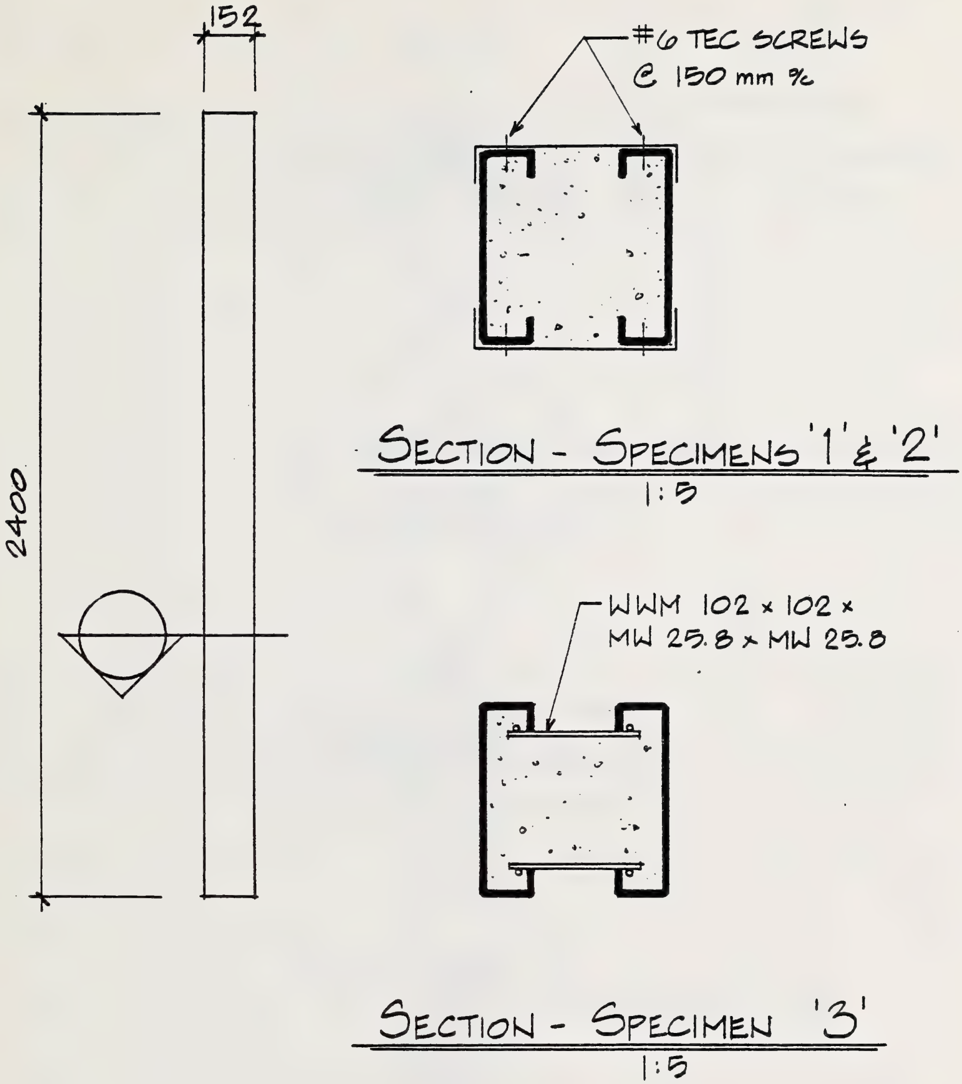
4.2.2.1 Column Test Specimens

Three full scale column specimens were fabricated at Rocky Mountain Precast using two methods. The first two used 152 mm 20 gauge (0.76 mm) channels and cover tracks on the side screwed together with number 6 Tec Screws at 150 mm o/c as shown in Figure 4.12. The third specimen was cast with wood forms on two sides screwed to the cold-formed channels and using 100 x 100 MW 25.8/MW 25.8 welded wire mesh to replace the cover tracks. The three column specimens were cast as they would be in the field with no difficulty.

4.2.2.2 Column Experimental Procedure

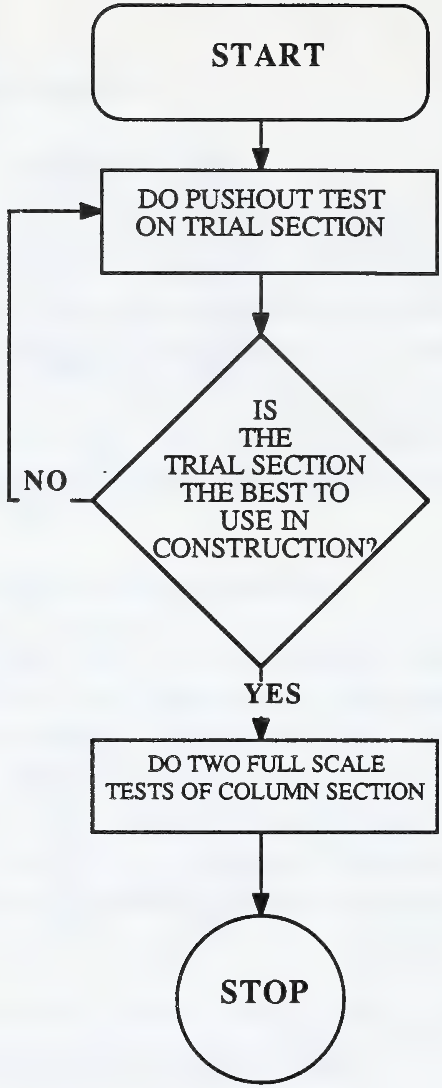
Figures 4.13 and 4.14 show the test procedure and setup used. The columns for the Gemini System were tested in two stages. The first stage used eighteen pushout and stub column tests to establish the best configuration for a composite column section for the second stage. The second stage had three full scale tests to confirm the capacities of 2400 mm long columns. The columns were loaded to failure with deflections being measured at the top and at mid-height of the columns using dial gauges. The columns were tested by Hardy-BBT Ltd. in Calgary.

Column test results and their analysis are discussed in Section 5.



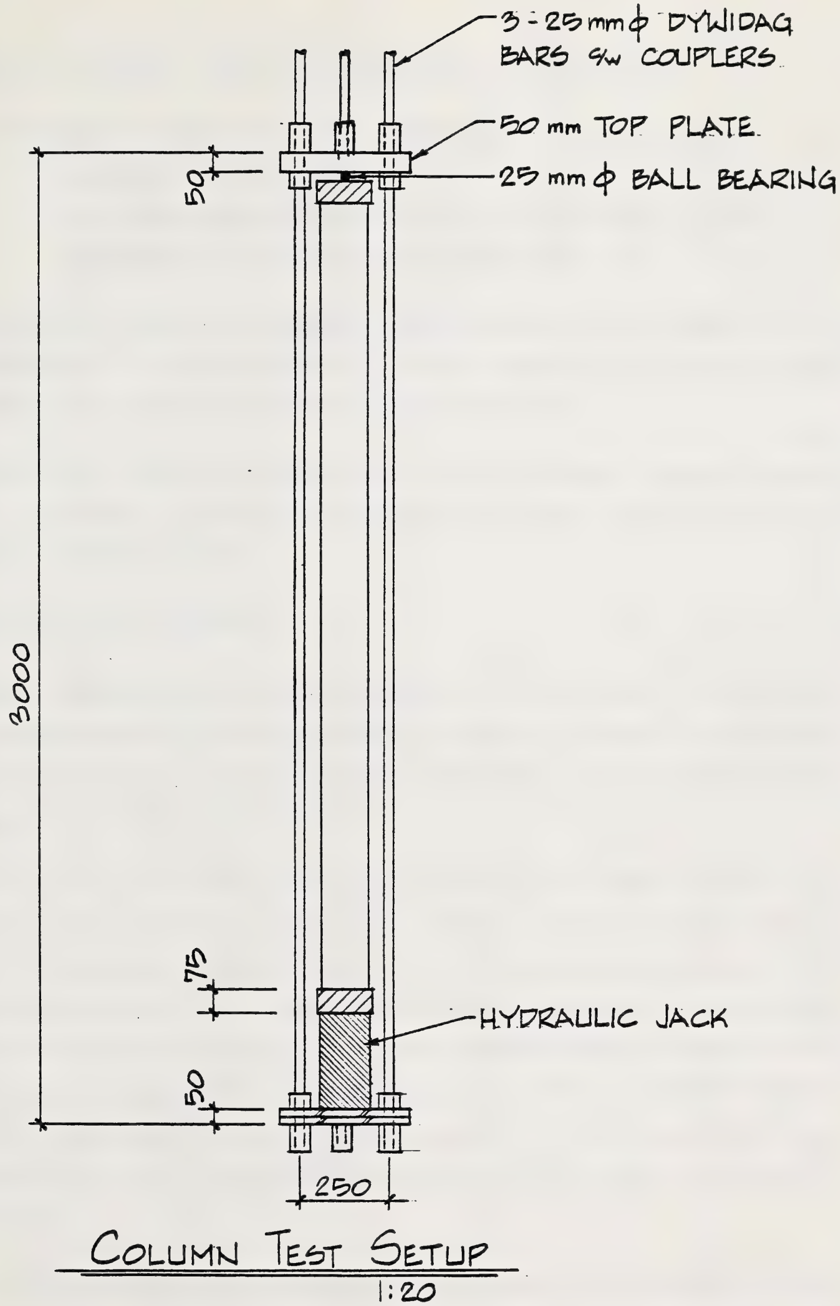
Column Specimens

Figure 4.12



COLUMN LOAD TEST FLOW DIAGRAM

FIGURE 4.13



Test Setup for Column

Figure 4.14

5.0 OBSERVATIONS AND DISCUSSION OF RESULTS

As was discussed in Section 4, the objectives of the test program were:

1. to ensure composite action, and
2. to validate the engineering models used to yield the data necessary for generation of the load tables.

In view of these objectives, particular attention was given to deflections in the verification of composite action, to moment capacity for beam and joist specimens, and to axial capacity for column specimens.

This section discusses sources of error associated with the field testing program, the results of the field testing, and how these results relate to the generation of the load tables.

5.1 SOURCES OF ERROR

A number of factors, that could be eliminated or reduced in a laboratory setting, contributed to possible variations in field test results. These were end support conditions, method of loading, deflection measurements, creep, shrinkage, and strength of concrete.

The end support used during testing of the first beam panel and both floor joist panels was a length of Hem Fir, 89 x 89 mm, which was screwed to the cold-form channels and laid on the ground. The wood supports were not attached to the ground in any way. If the supports prevented rotation of the channels at the centre of the supports and moved the point of rotation to the inside edge of the supports the moment capacity would be overestimated by 17 % and the deflections underestimated by 28 %. This potential error was eliminated on the second beam with the introduction of a pipe at one end to act as a roller.

The floor joist panels were loaded with 40 kg. bags of Crushed Dolomite aggregate. The variation in the weight of these bags, according to the manufacturer is 1 %. Thus the error caused in the test was 1 %. For the beam tests, when steel coils were used for loading and each coil was individually weighed.

Deflections were measured in two ways. The first method of measurement involved reading relative deflection changes using string lines and measuring tapes secured to the floor panels. These measurements were taken on both sides of the floor panel at midspan, and the quarter points. Deflections used in the analysis were an average of the two sides. There were 1 mm divisions on the tape and the error in readings was assumed to be 0.5 mm. A second set of measurements was taken during the tests using a tape and surveying level to confirm the results found using the string line. No discrepancies were found.

The most difficult error to estimate was that associated with the creep and shrinkage properties of the concrete, because shrinkage and creep are difficult to measure and calculate. No initial deflection was observed that could be attributed to shrinkage and calculations indicated that any shrinkage that could have occurred during the test would be negligible. Because of the lack of control of the test environment creep was ignored. The concrete strengths were confirmed using Schmit hammer tests which are only accurate to within 10 %. This variability would cause 5 % error in the calculation of the modulus of elasticity of the concrete, and a 3 % error in the moment capacity calculated using the S16.1 model. Errors in calculating deflections and moments using the yielding of the bottom flange were less than 1%.

It was concluded that the effect of errors on test measurements for the first beam, and both floor joist tests, would result in a maximum overestimation of moment induced by the imposed loads of 20 %, and a underestimation of 28 % for deflections.

Errors in strain calculations for the full scale column tests are dependent on the modulus of elasticity and areas of the Dywidag bars. The modulus of elasticity were considered to be within 10 % of their published values, and the resultant error on strain values was considered to be up to 5 %.

5.2 FLOOR JOISTS

The two goals in the floor joist experiments were:

1. to establish that the floor system behaved compositely, and
2. to develop an acceptable moment capacity model to be used in conjunction with S-136 to generate load tables.

The stiffness of the composite floor system was compared to the recommended equation of the Canadian Institute of Steel Construction for composite design, and the moment capacity was compared with the two possible moment models that could be used. The first moment model assumed that the member can only be loaded until the bottom flange of the cold-formed channel yields, while the second (presently used by S16.1) assumed that all of the cold-formed channel can yield.

The load deflection curves for the two floor joist panel tests are shown in Figures 5.1 and 5.2, with a comparison of predicted versus measured results in Table 5.1. The load deflection curves, and table, indicate that the moment capacities of the floor joists lie between the first yielding of the bottom flange of the channel (used in Elastic Design) and the full plastic section (as used in CAN3-S16.1). Table 5.1 also indicates that the floor joists behave compositely, and that the deflection can be calculated as recommended by the Canadian Institute of Steel Construction. Due to the large sources of error detailed in Section 5.1 and the extensive additional experimentation required, the decision was made to limit the ultimate moment capacities to the first yielding of the bottom flange. Analysis of the test results is contained in Appendix A, and additional information on the capacities of the shear tabs is given in Appendix D.

Ratio of Measured to Predicted Floor Joist Results
Table 5.1

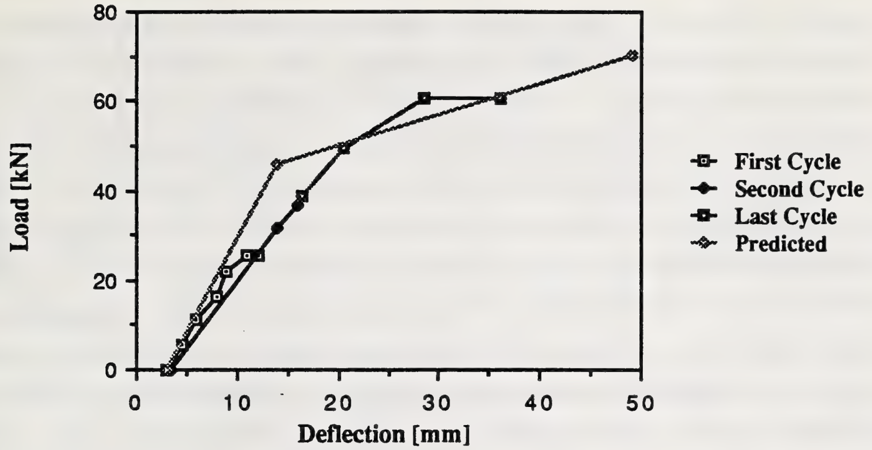
Test	Moment [S 16.1] Measured / Predicted	Moment [Elastic] Measured / Predicted	Deflection [Composite] Measured / Predicted
First	0.86	**1.31**	1.17
Second	1.14	**1.88**	1.03

5.3 FLOOR BEAMS

The three goals in the floor beam experiments were:

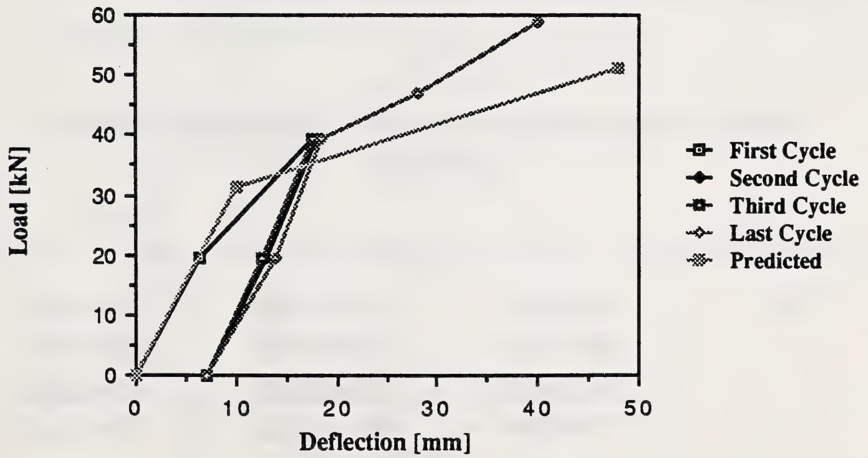
1. to establish that the floor beams behaved compositely,
2. to develop an acceptable moment capacity model, and
3. to check the shear capacity to be used to generate load tables.

The moment and shear capacities were compared to a plastic truss model, as used in A 23.3, which assumes full yielding of the cold-formed steel channels.



Floor Joist Panel I

Figure 5.1



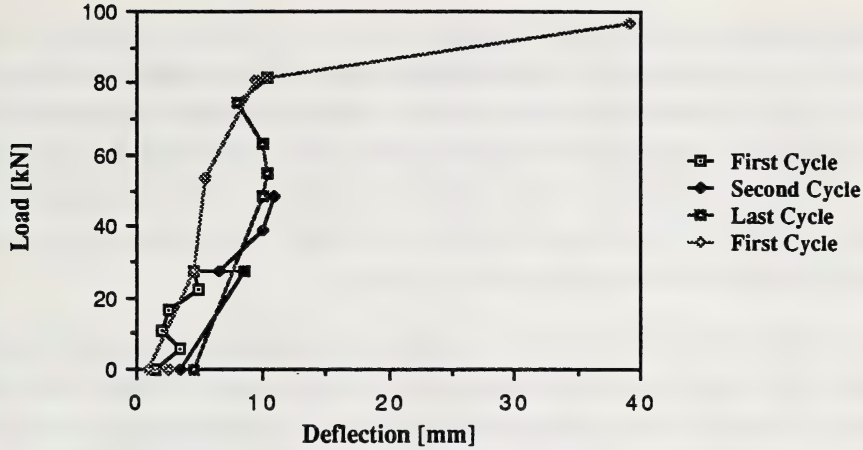
Floor Joist Panel II

Figure 5.2

Figures 5.3 and 5.4 show the load deflection curves of the two beam tests. The small deflections under the imposed loads indicate that the beams behaved compositely. Instrumentation results from the second test confirmed the results of the first test, in that the moment capacity of the beam can be calculated using the moment equations from CAN3-A23.3, taking d as the distance from the top of the beam to centroid of the beam channel. Strain gauge readings indicated that that beam channels had fully yielded across the full channel section. Neither test failed in shear. Test results were inconclusive as to whether the shear capacity can be modelled as a plastic truss, but indicated that the shear capacity was greater than that of the plain concrete or the cold-formed steel channels individually. The higher measured shear capacity of the beam could be explained assuming the beam acted as a reinforced concrete beam with the floor joists acting as shear stirrups (plastic truss), or, the concrete between the beam channels acted as web stiffeners thereby increasing the shear capacity of the channels. A summary of test results is shown in Table 5.2. The analysis and results of beam tests can be found in Appendix B.

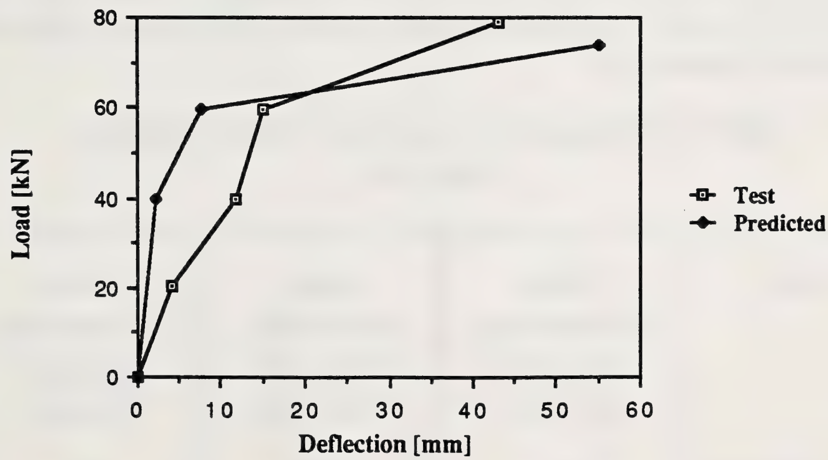
Ratio of Measured to Predicted Floor Beam Results
Table 5.2

Test	**Moment [A-23.3]** $\frac{\text{Measured}}{\text{Predicted}}$	**Moment [Elastic]** $\frac{\text{Measured}}{\text{Predicted}}$	**Shear [S-136]** $\frac{\text{Measured}}{\text{Predicted}}$
First	**0.91**	1.21	>1.52
Second	**1.05**	1.25	>1.43



Floor Beam Panel I

Figure 5.3



Floor Beam Panel II

Figure 5.4

5.4 COLUMNS

The two goals in the column tests were:

1. to establish that the columns behaved compositely, and
2. to develop an axial capacity model to generate load tables.

The axial capacity can be analyzed three ways: first, as a plain concrete column, second, using the cold-formed channels to confine the concrete and finally, by assuming composite action between the steel and concrete. These three models are compared in Table 5.3

Test results indicated that the columns acted compositely, confirming the results of a paper on axial capacity by George Abdel-Sayed and Kwok-Cheung Chung in the Canadian Journal of Civil Engineering, volume 14, 1987, but indicated that axial strains will approach 0.003 at failure. The column enclosed on the sides with the cover track, and the column with the wire mesh had equal axial capacity, but the enclosed column had a more desirable ductile behaviour. All the full-scale column specimens failed with crushing of concrete at the bottom of the columns. The location of the failure indicated that wider spacing of screws may be possible through the middle section of the columns. Analysis and results of the column tests are contained in Appendix C.

Ratio of Measured to Predicted Column Results
Table 5.3

Tests	**Axial [Unconfined]** Measured / Predicted	**Axial [Confined]** Measured / Predicted	**Axial [Composite]** Measured / Predicted
Short Column A	1.11	1.06	**.97**
Short Column B	1.12	1.12	**.98**
Long Column 1	1.25	1.20	**1.01**
Long Column 2	1.09	1.04	**.87**
Long Column 3	1.32	1.27	**1.01**

5.5 CONCLUSION

The test program established that the members of the Gemini System II act compositely. Further, the program established, or confirmed, models for moment, shear, and axial capacities that will permit generation of load tables to be used in design. These load tables are presented in the next section.

6.0 DEVELOPMENT OF LOAD TABLES

The test program described in sections 4 and 5 established or confirmed models for moment, shear, and axial capacities necessary for generation of load tables. The development of the load tables is presented in this section. The section is broken into three subsections, one for floor joists, the second, for floor beams, and the third, for columns. Each of these subsections is again broken down into three parts. The three parts are a specification which contains assumptions used in the generation of the load tables, a sample calculation, and load tables in both imperial and metric units. Governing assumptions and methods of construction, are presented in three-part specification format for ease of use by the consultant or contractor. The construction methods (shoring and concrete mix designs) are to be followed to obtain the design capacities stated in the load tables.

6.1 GEMINI LOST FORM JOIST LOAD TABLES

6.1.1 Joist Load Table Specifications

1.0 Load Table Calculations

.1 These tables were completed following accepted engineering practice and experimental results.

1.1 The degree of composite action is calculated following clauses 17.4.4 to 17.4.6 of CAN3-S16.1-M78

1.2 The effective width used is in accordance with clause 17.3.2.1 of CAN3-S16.1-M78

1.3 The effective Moment of Inertia is calculated in accordance with recommendations of the Canadian Institute of Steel Construction

1.4 These tables were calculated on the assumption that the joists will be shored at 2400 o/c during construction

1.5 The effect of creep on the deflection of the floor joist is calculated using $Ø_{creep}$ = 2.35, ACI Recommendation, K_{30} = .86.
Creep transformed Ec= Ec_{28} /2.5.

.2 The ultimate capacity of the joists is limited by the following criteria:

2.1 The moment capacity of the section is based on clause 6.4.1 (d) of CAN3-S136-M84 using an Elastic Cracked Transformed Section.

2.2 The Shear Capacity is limited using CAN3-S136-M84 clause 6.4.5.

.3 The serviceability capacity of the joist system is limited by two criteria.

3.1 Deflection is limited to a total deflection of L/240 using a creep transformed section.

3.2 Spans are limited to those with a natural frequency greater than 4 Hz . This is calculated following the recommendations provided in the Handbook of Steel Construction (CISC) (1982).

2.0 Materials

.1 Floor Joists: Cold-formed channels fabricated by Bailey Metals or Mantane Construction Products under licence of Gemini Structural Systems. The Section properties used were taken from the Bailey Mantane product catalogue. The steel yield strength is to be at least 230 MPa. for steel thinner than .06", or 345 MPa for thicker steel.

.2 Concrete : 25 MPa. with a cement water ratio of 0.50. The concrete is be cured for seven days and is to be shored until the concrete strength achieves 20 MPa.

3.0 Use of the Tables

Two sets of tables are provided giving service and ultimate load capacities

.1 The property tables give the section properties from which the load capacities are calculated.

I Moment of Inertia

Y Distance from the neutral axis to the bottom of the steel joist.

Nc/Lc The number of tabs required to achieve full composite action. (This is also the span, in feet, required to obtain composite action)

Lvib Maximum Span that has a natural frequency higher than 4 Hz.

.2 Ultimate Load Table

2.1 The joist spacing is noted at the top. (S)

2.2 The load capacities provided in the tables are the factored capacities of the system. When checking capacities, include the self weight of the system.

.3 Service Load Table

3.1 The capacities provided in the table are the maximum total loads allowed by limiting the deflection to L/240. When checking this capacity include the self weight of the system. If capacities are desired for a L/360 deflection limit, multiply the values in the tables by two thirds.

3.2 V indicates spans where the floor system has too low a natural frequency

6.1.2 Sample Calculation of Composite Floor Joist Properties

6" x .036 Joist @ 300 mm o/c with 65 mm concrete topping

Properties

6" x .036" Joist			**Concrete Topping**		
$F_y =$	230 MPa		$f'c =$	25 MPa	
$E_s =$	203 GPa		$E_c =$	$5.000\sqrt{f'c} =$	25.25 GPa
$I_j =$	$1.83\ in^4 =$	$7.62 \times 10^5\ mm^4$	$t =$	65 mm	
$A_j =$	$.358\ in^2 =$	$231\ mm^2$	$b =$	300 mm	
$h_j =$	6.0 " =	152 mm	$n =$	$\frac{E_c}{E_s} = 8.04$	

<u>Amount of Concrete in Compression</u>

$$c = \frac{nA_j}{b}\left[\left[2 \times \left(t+9+\frac{h_j}{2}\right) \times \frac{b}{nA_j} + 1\right]^{1/2} - 1\right]$$

c = 37.3 mm

$y = t + 9 + 152 - c/2 =$ 207.3 mm

<u>Cracked Transformed Section for Strength Calculations</u>

Member	y mm	A mm^2	y ' mm	Ay'^2 mm^4 $\times 10^6$	I mm^4 $\times 10^6$	$Ay'^2 + I$ mm^4 $\times 10^6$
Topping	207.3	1393.6	18.7	.486	.162	.648
Channel	76.2	231	112.6	2.932	.762	3.693
Total	**188.6**	**1624.6**		3.418	.924	**4.341**

$I_t = 4.341 \times 10^6\ mm^4$ $\qquad y_t = 188.6$ mm

$A_t = 1625\ mm^4$

$S_{bt} = I_t / y_t = 23011\ mm^3$

Strength Capacities for Fully Composite Floor Joist

Mr_{comp} = $Øs \times Sb_t \times Fy$

= $.9 \times 23011 \times 230$

= 4.763 kNm

Vr = 4.35 kN

Strength Capacities for Composite Floor Joist at Different Spans

a) l = 6000 mm

See whether section is composite — Following S 16.1

$$n = \frac{Øs\ As\ Fy \times 2}{Øc\ Vtab}$$

Vtab values recommended in Appendix D

$$= \frac{.9 \times 231 \times 230 \times 2}{.80 \times 2700}$$

= 45

number of tabs = l / 150

= 40

Therefore section is partially composite p = 40/45 = .89

$$Mr = \frac{Ie}{It} \times Mr_{comp}$$

reduce moment Capacity

$Ie = Ij + .85\,(It - Ij)\,p^{.25}$ — from S 16.1

$Ie = .762 + .85\,(4.341 - .762)\,.89^{.25} \times 10^6$

$= 3.715 \times 10^6\ mm^4$

Mr = 4.077 kNm

$w_{mom} = Mr/l^2 \times 8/b = 4077000 / 6000^2 \times 8 / .300$

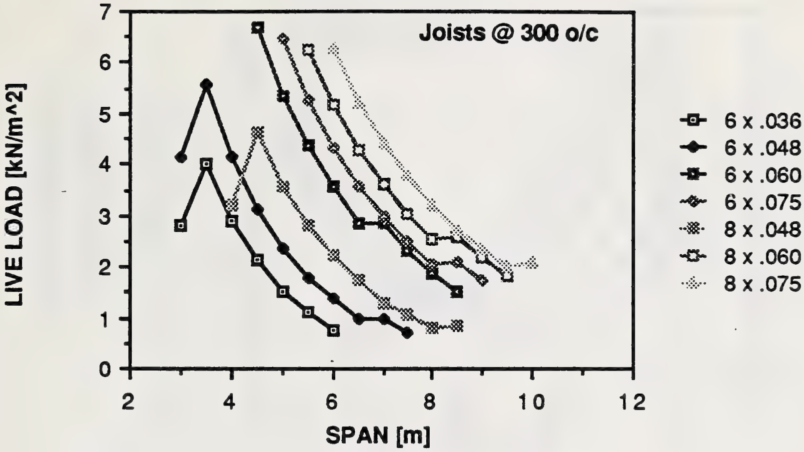
$= 3.01\ kN/m^2$

Vr = 4.35 kN

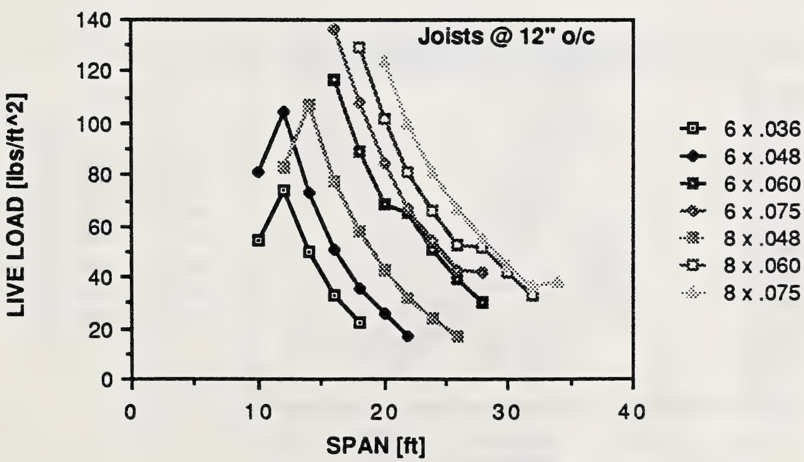
$w_{shear} = Vr/l \times 2/b = 4350 / 6000 \times 2 / .300$

$= 4.83\ kN/m^2$

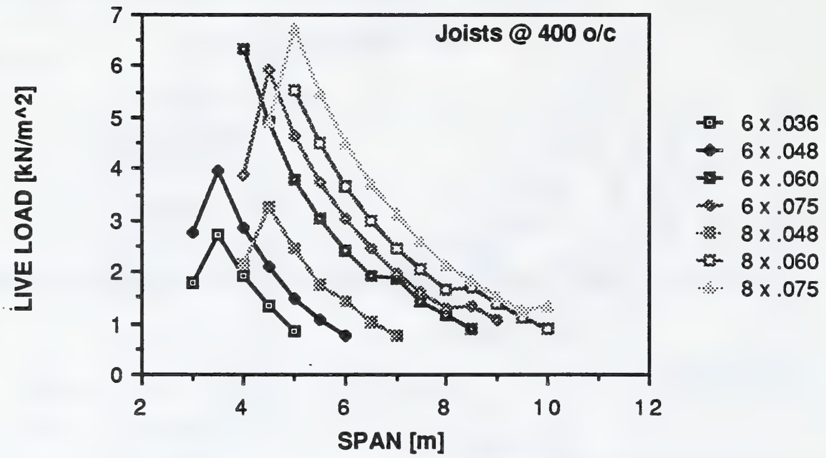
use $w = 3.01\ kN/m^2$



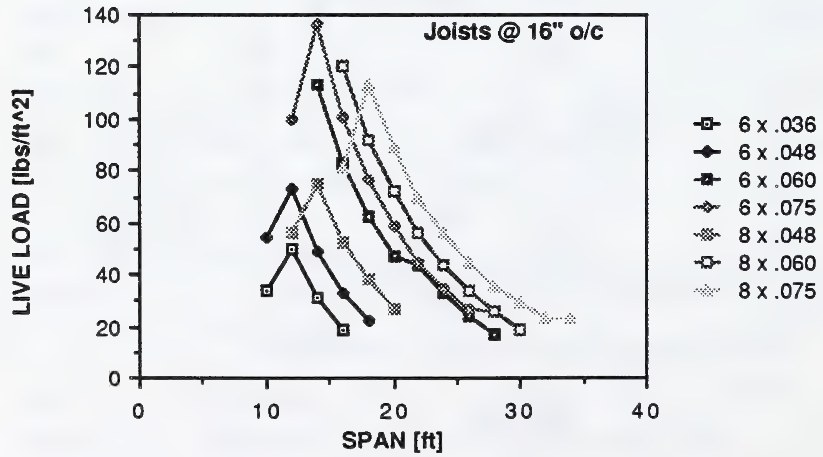
Live Load Span Chart
Figure 6.1



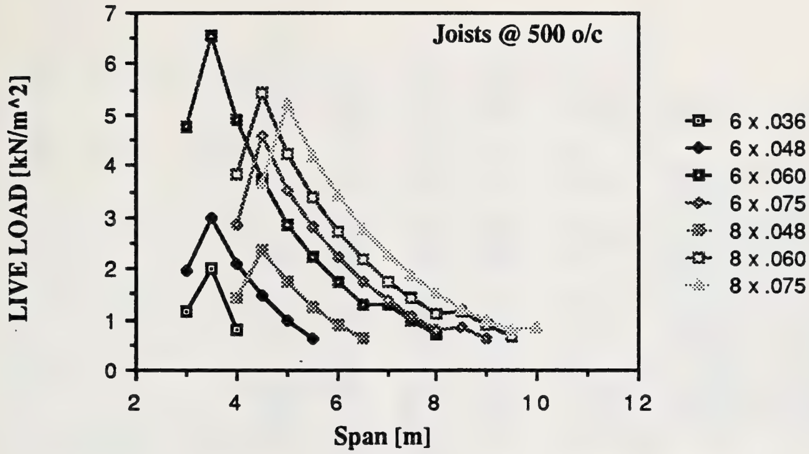
Live Load Span Chart
Figure 6.2



Live Load Span Chart
Figure 6.3

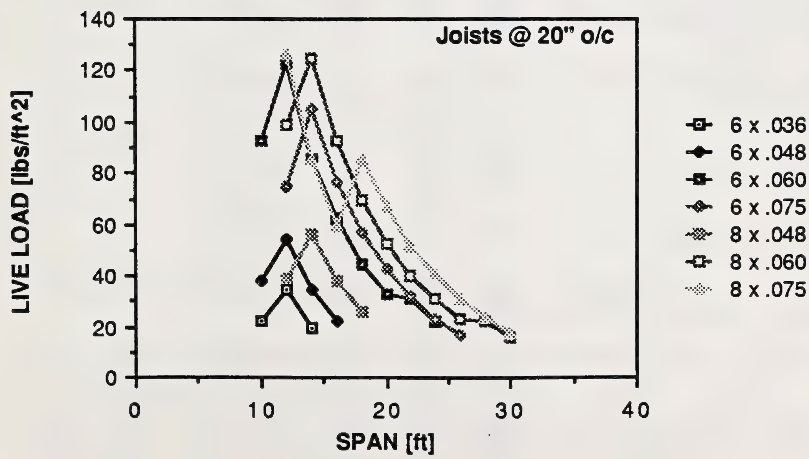


Live Load Span Chart
Figure 6.4



Live Load Span Chart

Figure 6.5



Live Load Span Chart

Figure 6.6

Joist Load Tables (Factored Loads)

Table 6.1

Section Properties of Lost Form System (65 mm Topping)

	Imperial Properties					Metric Properties				Composite Properties						
						Es= 203000 MPa				f'c =	25	37.352				
Channel	Thickness	Depth	Area	I	Mr	Depth	Area	I	Mr	y	Area	It	It-Is	Is	Nc/Lc	L vib
	in.	in.	in.^2	in.^4	in Kips	mm	mm^2	mm^4	kNm	mm	mm^2	mm^4	mm^4	mm^4	ft	mm
5.5 x .048	0.048	5.5	0.473	2.14	22.9	140	305	8.91E+05	2.59	173	1836	4.98E+06	4E+06	9E+05	23	8105
5.5 x .060	0.06	5.5	0.588	2.65	42.8	140	379	1.10E+06	4.84	169	2054	5.94E+06	5E+06	1E+06	22	8473
5.5 x .075	0.075	5.5	0.729	3.26	52.7	140	470	1.36E+06	5.95	165	2297	7.05E+06	6E+06	1E+06	28	8843
6 x .036	0.036	6	0.358	1.83	18.1	152	231	7.62E+05	2.04	189	1626	4.35E+06	4E+06	8E+05	23	7837
6 x .048	0.048	6	0.473	2.44	24.1	152	305	1.02E+06	2.72	184	1875	5.51E+06	4E+06	1E+06	23	8314
6 x .060	0.06	6	0.585	2.99	44.8	152	377	1.24E+06	5.06	180	2092	6.55E+06	5E+06	1E+06	22	8682
6 x .075	0.075	6	0.722	3.63	54.6	152	466	1.51E+06	6.17	176	2333	7.74E+06	6E+06	2E+06	27	9052
7.25 x .048	0.048	7.3	0.557	4.13	33.5	184	359	1.72E+06	3.78	210	2142	8.11E+06	6E+06	2E+06	27	9156
7.25 x .060	0.06	7.3	0.693	5.11	62.9	184	447	2.13E+06	7.11	206	2397	9.70E+06	8E+06	2E+06	26	9575
7.25 x .075	0.075	7.3	0.861	6.31	77.5	184	555	2.63E+06	8.76	201	2683	1.15E+07	9E+06	3E+06	33	10001
8 x .048	0.048	8	0.569	4.88	35.6	203	367	2.03E+06	4.02	227	2225	9.38E+06	7E+06	2E+06	28	9496
8 x .060	0.06	8	0.705	6	67.5	203	455	2.50E+06	7.63	223	2485	1.12E+07	9E+06	2E+06	27	9923
8 x .075	0.075	8	0.872	7.36	82.6	203	563	3.06E+06	9.33	218	2774	1.33E+07	1E+07	3E+06	33	10356
9.25 x .060	0.06	9.3	0.813	9.24	85.6	235	525	3.85E+06	9.67	248	2789	1.56E+07	1E+07	4E+06	31	10784
9.25 x .075	0.075	9.3	1.01	11.4	110	235	652	4.75E+06	12.4	243	3122	1.86E+07	1E+07	5E+06	38	11266
10 x .060	0.06	10	0.825	10.4	87.3	254	532	4.33E+06	9.86	265	2960	1.75E+07	1E+07	4E+06	31	11099
10 x .075	0.075	10	1.02	12.8	115	254	658	5.33E+06	13	259	3086	2.08E+07	2E+07	5E+06	39	11587
12 x .060	0.06	12	0.945	16.5	101	305	610	6.87E+06	11.4	307	3038	2.59E+07	2E+07	7E+06	36	12247
12 x .075	0.075	12	1.17	20.3	145	305	755	8.45E+06	16.4	300	3183	3.08E+07	2E+07	8E+06	44	12787

Ultimate Capacities Based on Cracked Elastic Properties and Unperforated Steel Joists

Section Properties of Lost Form System (65 mm Topping)

s= 300 mm

Factored Loads Joists Are Capable of Supporting for Different Spans

(kN/m^2)

Span (mm)	3000	3500	4000	4500	5000	5500	6000	6500	7000	7500	8000	8500	9000	9500	10000	10500
5.5 x .048	7.67	9.86	7.80	6.33	5.19	4.39	3.77	3.24	3.25	2.83	2.49	V	V	V	V	V
5.5 x .060	14.33	18.27	14.44	11.73	9.62	8.13	6.98	6.00	5.95	5.18	4.55	V	V	V	V	V
5.5 x .075	17.64	12.96	9.92	13.64	11.19	9.45	8.11	6.97	6.12	5.42	4.80	4.90	V	V	V	V
6 x .036	6.06	7.86	6.22	5.05	4.14	3.51	3.01	2.59	2.59	2.26	V	V	V	V	V	V
6 x .048	8.07	10.25	8.10	6.58	5.40	4.56	3.91	3.37	3.37	2.93	2.58	V	V	V	V	V
6 x .060	15.00	18.87	14.91	12.11	9.93	8.40	7.20	6.20	6.14	5.35	4.70	4.16	V	V	V	V
6 x .075	18.28	13.43	10.28	14.10	11.56	9.77	8.38	7.21	6.32	5.60	4.96	5.03	4.49	V	V	V
7.25 x .048	11.21	8.24	6.31	8.30	6.80	5.74	4.92	4.23	3.71	3.29	2.91	2.94	2.63	V	V	V
7.25 x .060	21.06	15.47	18.94	15.36	12.58	10.63	9.10	7.83	6.86	6.07	6.09	5.40	4.81	4.32	V	V
7.25 x .075	25.94	19.06	14.59	11.53	14.73	12.43	10.65	9.15	8.02	7.09	6.28	5.64	5.09	4.60	4.75	V
8 x .048	11.92	8.76	6.70	8.84	7.24	6.12	5.24	4.50	3.95	3.50	3.09	3.15	2.81	V	V	V
8 x .060	22.60	16.60	12.71	16.28	13.34	11.26	9.65	8.29	7.27	6.43	5.70	5.75	5.13	4.60	V	V
8 x .075	27.65	20.31	15.55	12.29	15.65	13.21	11.31	9.72	8.52	7.53	6.67	5.99	5.41	4.88	5.04	V
9.25 x .060	28.65	21.05	16.12	12.74	16.42	13.85	11.86	10.19	8.93	7.89	6.99	6.27	5.66	5.76	5.20	4.72
9.25 x .075	36.82	27.05	20.71	16.37	13.26	10.96	13.98	12.01	10.52	9.30	8.23	7.38	6.67	6.02	5.49	5.03
10 x .060	27.00	21.47	16.44	12.99	17.29	14.58	12.48	10.72	9.40	8.31	7.35	6.60	5.96	6.07	5.47	4.96
10 x .075	38.50	28.28	21.65	17.11	13.86	11.45	14.60	12.54	10.99	9.71	8.59	7.71	6.96	6.28	5.73	5.25
12 x .060	22.53	19.33	16.90	15.03	12.17	18.28	15.64	13.43	11.76	10.39	9.20	8.25	7.45	6.72	6.13	5.62
12 x .075	44.00	35.66	27.30	21.57	17.47	14.44	12.13	10.34	13.88	12.26	10.84	9.72	8.77	7.92	7.22	6.62

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Ultimate Capacities Based on Cracked Elastic Properties and Unperforated Steel Joists

Section Properties of Lost Form System [65 mm Topping]

s= 400 mm

Factored Loads Joists Are Capable of Supporting for Different Spans

[kN/m^2]

Span [mm]	3000	3500	4000	4500	5000	5500	6000	6500	7000	7500	8000	8500	9000	9500	10000	10500
5.5 x .048	5.75	7.52	5.95	4.83	3.96	3.35	2.88	2.48	2.48	2.16	1.90	V	V	V	V	V
5.5 x .060	10.75	13.95	11.03	8.97	7.35	6.22	5.34	4.59	4.56	3.97	3.49	3.09	V	V	V	V
5.5 x .075	13.23	9.72	7.44	10.44	8.56	7.24	6.21	5.34	4.69	4.15	3.68	3.77	V	V	V	V
6 x .036	4.54	5.99	4.73	3.85	3.16	2.67	2.29	1.97	1.98	1.72	V	V	V	V	V	V
6 x .048	6.05	7.81	6.18	5.02	4.12	3.48	2.99	2.57	2.58	2.24	1.97	V	V	V	V	V
6 x .060	11.25	14.41	11.40	9.26	7.59	6.42	5.51	4.74	4.70	4.10	3.60	3.19	V	V	V	V
6 x .075	13.71	10.07	7.71	10.79	8.85	7.48	6.42	5.52	4.85	4.29	3.80	3.86	3.45	V	V	V
7.25 x .048	8.41	6.18	4.73	6.33	5.19	4.38	3.76	3.23	2.83	2.51	2.22	2.25	2.01	V	V	V
7.25 x .060	15.79	11.60	14.46	11.73	9.61	8.12	6.96	5.98	5.25	4.65	4.67	4.13	3.69	3.31	V	V
7.25 x .075	19.46	14.30	10.94	8.65	11.26	9.51	8.15	7.00	6.14	5.43	4.81	4.32	3.90	3.52	3.65	V
8 x .048	8.94	6.57	5.03	6.74	5.52	4.66	4.00	3.44	3.01	2.67	2.36	2.41	2.15	1.93	V	V
8 x .060	16.95	12.45	9.53	12.43	10.19	8.60	7.37	6.34	5.56	4.92	4.36	4.41	3.93	3.53	3.18	V
8 x .075	20.74	15.24	11.67	9.22	11.96	10.10	8.65	7.44	6.52	5.77	5.11	4.59	4.14	3.74	3.87	V
9.25 x .060	21.49	15.79	12.09	9.55	12.53	10.57	9.05	7.78	6.82	6.03	5.34	4.79	4.33	4.42	3.99	3.62
9.25 x .075	27.62	20.29	15.53	12.27	9.94	8.22	10.69	9.18	8.05	7.11	6.29	5.65	5.10	4.61	4.20	3.85
10 x .060	20.25	16.10	12.33	9.74	13.23	11.16	9.55	8.21	7.20	6.36	5.63	5.06	4.57	4.66	4.21	3.81
10 x .075	28.87	21.21	16.24	12.83	10.39	8.59	11.16	9.59	8.40	7.43	6.57	5.90	5.33	4.81	4.39	4.02
12 x .060	16.90	14.50	12.68	11.27	9.13	13.94	11.93	10.25	8.97	7.93	7.02	6.30	5.69	5.13	4.69	4.29
12 x .075	33.00	26.75	20.48	16.18	13.11	10.83	9.10	7.75	10.58	9.35	8.27	7.42	6.70	6.04	5.51	5.05

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ALBERTA
David P. Thompson
December 15, 1989

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Ultimate Capacities Based on Cracked Elastic Properties and Unperforated Steel Joists

Section Properties of Lost Form System (65 mm Topping)

s= 500 mm

Factored Loads Joists Are Capable of Supporting for Different Spans

[kN/m^2]

Span (mm)		3000	3500	4000	4500	5000	5500	6000	6500	7000	7500	8000	8500	9000	9500	10000	10500
	|||																
5.5 x .048	|||	4.60	6.09	4.82	3.91	3.21	2.72	2.33	2.01	2.01	1.75	1.54	V	V	V	V	V
5.5 x .060	|||	8.60	11.31	8.94	7.27	5.96	5.05	4.33	3.73	3.70	3.23	2.84	2.51	V	V	V	V
5.5 x .075	|||	10.58	7.78	5.95	8.47	6.95	5.88	5.04	4.34	3.81	3.37	2.99	3.06	2.73	V	V	V
	|||																
6 x .036	|||	3.64	4.84	3.83	3.11	2.55	2.16	1.85	1.60	1.60	1.40	V	V	V	V	V	V
6 x .048	|||	4.84	6.32	5.00	4.06	3.33	2.82	2.42	2.08	2.09	1.82	1.60	V	V	V	V	V
6 x .060	|||	9.00	11.68	9.24	7.50	6.16	5.21	4.47	3.84	3.82	3.33	2.92	2.59	V	V	V	V
6 x .075	|||	10.97	8.06	6.17	8.76	7.18	6.07	5.21	4.48	3.93	3.48	3.09	3.14	2.80	V	V	V
	|||																
7.25 x .048	|||	6.73	4.94	3.78	5.12	4.20	3.55	3.04	2.62	2.30	2.03	1.80	1.83	1.63	V	V	V
7.25 x .060	|||	12.63	9.28	11.71	9.50	7.79	6.58	5.64	4.85	4.26	3.77	3.79	3.36	3.00	2.69	V	V
7.25 x .075	|||	15.57	11.44	8.76	6.92	9.13	7.71	6.61	5.68	4.98	4.41	3.90	3.51	3.17	2.86	2.97	V
	|||																
8 x .048	|||	7.15	5.25	4.02	5.45	4.47	3.78	3.24	2.78	2.44	2.16	1.91	1.95	1.74	1.56	V	V
8 x .060	|||	13.56	9.96	7.63	10.07	8.25	6.97	5.98	5.14	4.51	3.99	3.53	3.58	3.19	2.86	2.59	V
8 x .075	|||	16.59	12.19	9.33	7.37	9.70	8.19	7.02	6.03	5.29	4.68	4.14	3.72	3.36	3.04	3.15	2.85
	|||																
9.25 x .060	|||	17.19	12.63	9.67	7.64	10.15	8.57	7.33	6.31	5.53	4.89	4.33	3.89	3.51	3.59	3.24	2.94
9.25 x .075	|||	22.09	16.23	12.43	9.82	7.95	6.57	8.66	7.45	6.53	5.77	5.11	4.58	4.14	3.74	3.41	3.13
	|||																
10 x .060	|||	16.20	12.88	9.86	7.79	10.76	9.08	7.78	6.69	5.86	5.18	4.59	4.12	3.72	3.80	3.43	3.11
10 x .075	|||	23.10	16.97	12.99	10.27	8.32	6.87	9.07	7.79	6.83	6.04	5.34	4.80	4.33	3.91	3.57	3.27
	|||																
12 x .060	|||	13.52	11.60	10.14	9.02	7.30	11.30	9.67	8.31	7.28	6.44	5.70	5.11	4.62	4.17	3.80	3.49
12 x .075	|||	26.40	21.40	16.38	12.94	10.48	8.66	7.28	6.20	8.58	7.58	6.71	6.02	5.43	4.90	4.47	4.10

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Ultimate Capacities Based on Cracked Elastic Properties and Unperforated Steel Joists

Section Properties of Lost Form System [2.5" Topping]

s= 12 Inches

Factored Loads Joists Are Capable of Supporting for Different Spans

[psf]

Span (ft)	10	12	14	16	18	20	22	24	26	28	30	32	34	36	38	40
5.5 x .048	153	189	143	112	91	75	63	61	52	V	V	V	V	V	V	V
5.5 x .060	285	350	265	208	168	139	133	112	96	V	V	V	V	V	V	V
5.5 x .075	351	244	308	242	196	162	136	116	101	100	V	V	V	V	V	V
6 x .036	121	151	114	90	73	60	51	49	V	V	V	V	V	V	V	V
6 x .048	161	196	149	117	94	78	66	63	54	V	V	V	V	V	V	V
6 x .060	299	362	274	215	174	144	138	116	99	85	V	V	V	V	V	V
6 x .075	364	253	319	250	202	167	141	120	104	103	V	V	V	V	V	V
7.25 x .048	223	155	188	147	119	98	83	71	61	60	52	V	V	V	V	V
7.25 x .060	419	291	347	272	220	182	153	130	128	110	96	V	V	V	V	V
7.25 x .075	517	359	264	202	257	212	178	162	132	115	102	90	V	V	V	V
8 x .048	237	165	200	157	126	104	88	75	65	64	56	V	V	V	V	V
8 x .060	450	313	368	289	233	192	162	138	119	117	102	90	V	V	V	V
8 x .075	551	382	281	215	273	225	190	162	140	122	108	96	97	V	V	V
9.25 x .060	571	396	291	355	287	236	199	170	147	128	113	113	100	V	V	V
9.25 x .075	733	509	374	287	226	279	234	200	173	151	133	118	106	95	V	V
10 x .060	555	404	297	374	302	249	209	178	154	135	119	118	105	94	V	V
10 x .075	767	532	391	300	237	291	245	209	180	157	139	123	110	99	90	V
12 x .060	463	398	347	263	378	312	262	223	193	168	148	132	118	120	107	97
12 x .075	905	671	493	378	298	242	309	263	227	199	175	155	139	125	114	103

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PROFESSIONAL ENGINEER
D. P. THOMPSON
ALBERTA
David P Thompson
December 15/1989

Ultimate Capacities Based on Cracked Elastic Properties and Unperforated Steel Joists

Section Properties of Lost Form System [2.5" Topping]

s= 16 inches

Factored Loads Joists Are Capable of Supporting for Different Spans

[psf]

Span [ft]	10	12	14	16	18	20	22	24	26	28	30	32	34	36	38	40
5.5 x .048	115	144	109	86	69	57	48	47	40	V	V	V	V	V	V	V
5.5 x .060	214	267	202	159	129	106	102	86	73	63	V	V	V	V	V	V
5.5 x .075	264	183	236	185	150	124	104	89	77	77	V	V	V	V	V	V
6 x .036	91	115	87	68	55	46	38	37	V	V	V	V	V	V	V	V
6 x .048	121	150	113	89	72	60	50	49	41	V	V	V	V	V	V	V
6 x .060	224	276	209	164	133	110	105	89	76	65	V	V	V	V	V	V
6 x .075	273	190	244	192	155	128	108	92	80	79	69	V	V	V	V	V
7.25 x .048	168	116	143	112	91	75	63	54	47	46	40	V	V	V	V	V
7.25 x .060	315	218	265	208	168	139	117	100	98	84	74	V	V	V	V	V
7.25 x .075	388	269	198	151	197	162	137	117	101	88	78	69	V	V	V	V
8 x .048	178	124	152	119	96	80	67	57	50	49	43	V	V	V	V	V
8 x .060	338	234	281	220	178	147	124	106	91	90	78	69	V	V	V	V
8 x .075	413	287	211	161	209	172	145	124	107	94	83	73	74	V	V	V
9.25 x .060	428	297	218	271	219	180	152	130	112	98	86	86	76	V	V	V
9.25 x .075	550	382	281	215	170	213	179	153	132	115	102	90	81	73	V	V
10 x .060	416	303	223	286	231	191	160	137	118	103	91	91	81	72	V	V
10 x .075	575	399	293	225	177	223	187	160	138	120	106	94	85	76	69	V
12 x .060	347	298	261	197	288	238	200	170	147	129	113	101	90	92	82	74
12 x .075	679	504	370	283	224	181	236	201	173	151	134	119	106	96	87	79

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Ultimate Capacities Based on Cracked Elastic Properties and Unperforated Steel Joists

Section Properties of Lost Form System (2.5" Topping)

s= 20 inches

Factored Loads Joists Are Capable of Supporting for Different Spans

(psf)

Span (ft)		10	12	14	16	18	20	22	24	26	28	30	32	34	36	38	40
5.5 x .048	|||	92	117	88	69	56	46	39	38	32	V	V	V	V	V	V	V
5.5 x .060	|||	171	217	164	129	104	86	83	70	59	51	V	V	V	V	V	V
5.5 x .075	|||	211	146	191	150	122	101	85	72	63	63	V	V	V	V	V	V
6 x .036	|||	72	93	70	55	45	37	31	30	26	V	V	V	V	V	V	V
6 x .048	|||	96	121	92	72	58	48	41	39	34	V	V	V	V	V	V	V
6 x .060	|||	179	224	169	133	108	89	86	72	61	53	V	V	V	V	V	V
6 x .075	|||	218	152	198	155	126	104	87	75	65	64	56	V	V	V	V	V
7.25 x .048	|||	134	93	116	91	73	61	51	44	38	37	32	V	V	V	V	V
7.25 x .060	|||	252	175	215	169	136	112	95	81	80	69	60	52	V	V	V	V
7.25 x .075	|||	310	215	158	121	160	132	111	95	82	72	63	56	V	V	V	V
8 x .048	|||	142	99	123	97	78	65	54	46	40	40	35	V	V	V	V	V
8 x .060	|||	270	188	227	179	144	119	100	86	74	73	64	56	V	V	V	V
8 x .075	|||	330	229	169	129	169	140	118	101	87	76	67	60	60	V	V	V
9.25 x .060	|||	342	238	175	220	177	146	123	105	91	79	70	70	62	55	V	V
9.25 x .075	|||	440	306	225	172	136	173	145	124	107	94	83	73	66	59	V	V
10 x .060	|||	333	243	178	233	188	155	130	111	96	84	74	74	66	59	V	V
10 x .075	|||	460	319	235	180	142	181	152	130	112	98	86	77	69	62	56	V
12 x .060	|||	278	239	208	158	234	193	162	138	119	104	92	82	73	75	67	60
12 x .075	|||	543	403	296	227	179	145	191	163	141	123	108	96	86	78	70	64

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Ultimate Capacities Based on Cracked Elastic Properties and Unperforated Steel Joists

Joist Deflection Tables (Service Loads)

Table 6.2

Section Properties of Lost Form System (65 mm Topping)

s= 300 mm

Service Loads Joists Are Capable of Supprting for Different Spans

[kN/m^2]

Channel	3000	3500	4000	4500	5000	5500	6000	6500	7000	7500	8000	8500	9000	9500	10000	10500
5.5 x .048	24.97	15.99	11.04	7.96	5.87	4.51	3.54	2.81	2.59	2.11	V	V	V	V	V	V
5.5 x .060	29.69	19.01	13.12	9.46	6.97	5.35	4.20	3.34	3.05	2.48	2.04	V	V	V	V	V
5.5 x .075	33.39	21.37	14.73	10.60	7.82	6.00	4.71	3.73	3.04	2.51	2.08	V	V	V	V	V
6 x .036	22.25	14.26	9.85	7.10	5.24	4.03	3.16	2.51	2.32	1.89	V	V	V	V	V	V
6 x .048	27.81	17.81	12.29	8.86	6.54	5.02	3.94	3.13	2.88	2.35	V	V	V	V	V	V
6 x .060	32.90	21.07	14.54	10.48	7.73	5.93	4.66	3.70	3.37	2.74	2.26	V	V	V	V	V
6 x .075	37.10	23.74	16.37	11.78	8.69	6.66	5.23	4.15	3.38	2.79	2.31	2.19	V	V	V	V
7.25 x .048	40.49	25.91	17.85	12.84	9.47	7.26	5.69	4.52	3.67	3.03	2.52	2.38	V	V	V	V
7.25 x .060	48.24	30.85	21.24	15.28	11.26	8.63	6.77	5.37	4.37	3.60	3.36	2.80	2.36	V	V	V
7.25 x .075	54.44	34.79	23.92	17.19	12.66	9.69	7.59	6.02	4.89	4.03	3.35	2.82	2.41	V	V	V
8 x .048	46.81	29.94	20.62	14.84	10.93	8.38	6.57	5.21	4.24	3.50	2.90	2.76	2.32	V	V	V
8 x .060	55.31	35.37	24.34	17.50	12.90	9.88	7.75	6.15	5.00	4.12	3.42	3.22	2.71	V	V	V
8 x .075	62.80	40.13	27.59	19.82	14.60	11.18	8.76	6.94	5.64	4.65	3.86	3.26	2.78	2.37	V	V
9.25 x .060	76.54	48.90	33.60	24.12	17.76	13.60	10.65	8.44	6.86	5.65	4.69	3.95	3.37	3.22	2.76	V
9.25 x .075	87.54	55.88	38.35	27.50	20.24	15.48	12.11	9.60	7.79	6.42	5.32	4.49	3.82	3.27	2.83	2.47
10 x .060	86.33	55.15	37.89	27.21	20.04	15.33	12.01	9.52	7.74	6.38	5.29	4.46	3.80	3.63	3.11	2.69
10 x .075	98.09	62.61	42.97	30.81	22.68	17.34	13.57	10.75	8.73	7.19	5.96	5.03	4.28	3.66	3.17	2.76
12 x .060	******	80.49	55.23	39.61	29.15	22.29	17.44	13.82	11.22	9.24	7.66	6.46	5.50	4.70	4.07	3.55
12 x .075	******	91.93	62.99	45.12	33.19	25.35	19.82	15.70	12.74	10.48	8.69	7.32	6.23	5.33	4.61	4.02

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Deflection Capacities Based on Creep Transformed Properties [1/240]

Section Properties of Lost Form System [65 mm Topping]

s= 400 mm

Service Loads Joists Are Capable of Supprting for Different Spans

[kN/m^2]

Channel	3000	3500	4000	4500	5000	5500	6000	6500	7000	7500	8000	8500	9000	9600	10000	10500
5.5 x .048	19.92	12.76	8.82	6.36	4.69	3.61	2.83	2.25	2.08	1.69	V	V	V	V	V	V
5.5 x .060	23.81	15.25	10.54	7.60	5.61	4.30	3.38	2.68	2.46	2.00	1.65	V	V	V	V	V
5.5 x .075	26.87	17.20	11.87	8.55	6.31	4.84	3.80	3.02	2.46	2.03	1.68	1.61	V	V	V	V
6 x .036	17.62	11.29	7.81	5.63	4.16	3.19	2.51	1.99	1.85	1.50	V	V	V	V	V	V
6 x .048	22.15	14.19	9.80	7.07	5.22	4.01	3.15	2.50	2.31	1.88	1.55	V	V	V	V	V
6 x .060	26.34	16.87	11.65	8.40	6.20	4.76	3.74	2.97	2.72	2.21	1.82	V	V	V	V	V
6 x .075	29.81	19.09	13.17	9.49	7.00	5.37	4.22	3.35	2.72	2.25	1.87	1.77	V	V	V	V
7.25 x .048	32.17	20.59	14.20	10.22	7.54	5.78	4.54	3.60	2.93	2.42	2.01	1.90	V	V	V	V
7.25 x .060	38.50	24.64	16.98	12.22	9.01	6.91	5.42	4.30	3.50	2.89	2.70	2.25	1.90	V	V	V
7.25 x .075	43.64	27.90	19.21	13.81	10.18	7.80	6.11	4.85	3.94	3.25	2.70	2.28	1.94	1.66	V	V
8 x .048	37.14	23.77	16.38	11.79	8.69	6.67	5.23	4.15	3.38	2.79	2.31	2.21	1.86	V	V	V
8 x .060	44.11	28.22	19.44	13.99	10.31	7.91	6.20	4.92	4.00	3.30	2.74	2.59	2.18	1.86	V	V
8 x .075	50.31	32.17	22.14	15.92	11.73	8.99	7.04	5.59	4.54	3.75	3.11	2.62	2.24	1.91	V	V
9.25 x .060	61.01	39.00	26.82	19.28	14.20	10.87	8.52	6.76	5.49	4.53	3.76	3.17	2.70	2.59	2.22	V
9.25 x .075	70.07	44.76	30.75	22.08	16.25	12.44	9.74	7.72	6.27	5.17	4.29	3.62	3.08	2.63	2.28	1.99
10 x .060	68.81	43.98	30.25	21.74	16.02	12.27	9.61	7.62	6.20	5.11	4.24	3.58	3.05	2.92	2.51	2.16
10 x .075	78.51	50.14	34.45	24.73	18.21	13.93	10.91	8.65	7.03	5.79	4.80	4.05	3.45	2.95	2.56	2.23
12 x .060	######	64.22	44.11	31.66	23.31	17.84	13.97	11.07	8.99	7.41	6.15	5.19	4.42	3.78	3.27	2.85
12 x .075	######	73.65	50.53	36.23	26.66	20.39	15.95	12.64	10.26	8.45	7.00	5.91	5.03	4.30	3.72	3.25

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Deflection Capacities Based on Creep Transformed Properties [1/240]

Section Properties of Lost Form System [65 mm Topping]

s= 500 mm

Service Loads Joists Are Capable of Supprting for Different Spans

[kN/m^2]

Channel	3000	3500	4000	4500	5000	5500	6000	6500	7000	7500	8000	8500	9000	9500	10000	10500
5.5 x .048	16.65	10.67	7.38	5.32	3.93	3.02	2.37	1.88	1.75	1.42	V	V	V	V	V	V
5.5 x .060	19.97	12.80	8.85	6.38	4.71	3.62	2.84	2.26	2.07	1.68	1.39	V	V	V	V	V
5.5 x .075	22.61	14.48	10.00	7.21	5.32	4.08	3.21	2.54	2.07	1.71	1.42	1.36	V	V	V	V
6 x .036	14.64	9.39	6.49	4.69	3.46	2.66	2.09	1.66	1.54	1.25	V	V	V	V	V	V
6 x .048	18.49	11.85	8.19	5.91	4.36	3.35	2.63	2.09	1.94	1.57	1.30	V	V	V	V	V
6 x .060	22.07	14.14	9.77	7.05	5.20	4.00	3.14	2.49	2.29	1.86	1.53	1.28	V	V	V	V
6 x .075	25.05	16.05	11.08	7.98	5.89	4.52	3.55	2.82	2.30	1.90	1.57	1.50	V	V	V	V
7.25 x .048	26.82	17.17	11.84	8.53	6.29	4.83	3.79	3.01	2.45	2.02	1.68	1.59	V	V	V	V
7.26 x .060	32.20	20.61	14.21	10.24	7.55	5.79	4.54	3.61	2.94	2.42	2.27	1.89	1.59	V	V	V
7.25 x .075	36.59	23.41	16.12	11.60	8.55	6.55	5.14	4.08	3.32	2.74	2.27	1.92	1;64	1.40	V	V
8 x .048	30.92	19.80	13.65	9.83	7.25	5.56	4.37	3.46	2.82	2.33	1.93	1.85	1.55	V	V	V
8 x .060	36.85	23.59	16.26	11.71	8.63	6.62	5.19	4.12	3.35	2.77	2.30	2.18	1.83	1.56	V	V
8 x .075	42.15	26.96	18.57	13.36	9.85	7.55	5.92	4.70	3.82	3.15	2.61	2.21	1.88	1.61	1.57	V
9.25 x .060	50.93	32.56	22.41	16.12	11.87	9.10	7.13	5.66	4.60	3.79	3.15	2.66	2.27	2.18	1.87	1.61
9.25 x .075	58.68	37.50	25.78	18.52	13.64	10.44	8.18	6.49	5.27	4.35	3.60	3.04	2.59	2.22	1.92	1.68
10 x .060	57.43	36.72	25.28	18.17	13.39	10.26	8.04	6.38	5.19	4.28	3.55	3.00	2.56	2.46	2.11	1.82
10 x .075	65.73	42.00	28.88	20.74	15.28	11.70	9.16	7.27	5.90	4.87	4.04	3.41	2.90	2.48	2.15	1.88
12 x .060	83.92	53.62	36.86	26.47	19.50	14.93	11.69	9.27	7.53	6.21	5.15	4.35	3.70	3.17	2.74	2.39
12 x .075	96.64	61.71	42.37	30.40	22.38	17.12	13.40	10.62	8.63	7.11	5.89	4.97	4.23	3.62	3.14	2.73

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Deflection Capacities Based on Creep Transformed Properties [1/240]

Section Properties of Lost Form System [2.5" Topping]

s= 12 inches

Service Loads Joists Are Capable of Supporting for Different Spans

[psf]

Span [ft]	10	12	14	16	18	20	22	24	26	28	30	32	34	36	38	40
5.5 x .048	491	294	190	131	94	70	53	47	V	V	V	V	V	V	V	V
5.5 x .060	584	349	226	155	111	83	72	55	43	V	V	V	V	V	V	V
5.5 x .075	657	392	254	174	125	93	71	55	44	V	V	V	V	V	V	V
6 x .036	438	262	170	117	84	62	48	42	V	V	V	V	V	V	V	V
6 x .048	548	327	212	146	104	78	59	52	41	V	V	V	V	V	V	V
6 x .060	649	388	251	172	124	92	79	61	48	V	V	V	V	V	V	V
6 x .075	731	436	282	193	139	103	79	62	49	45	V	V	V	V	V	V
7.25 x .048	797	475	307	210	151	112	86	67	53	48	V	V	V	V	V	V
7.25 x .060	949	566	366	250	179	133	102	80	71	57	V	V	V	V	V	V
7.25 x .075	1071	638	412	282	202	150	114	89	71	58	47	V	V	V	V	V
8 x .048	921	549	355	243	174	129	99	77	62	56	V	V	V	V	V	V
8 x .060	1089	649	419	287	206	153	117	91	73	66	53	V	V	V	V	V
8 x .075	1239	737	476	325	233	173	132	103	82	67	55	46	V	V	V	V
9.25 x .060	1506	896	578	395	283	210	160	125	100	81	66	61	V	V	V	V
9.25 x .075	1723	1023	659	450	322	238	182	142	113	92	75	63	53	V	V	V
10 x .060	1698	1010	652	446	319	236	180	141	112	91	75	69	58	V	V	V
10 x .075	1930	1147	738	505	361	267	204	159	127	103	84	70	59	50	V	V
12 x .060	2481	1474	949	648	463	343	262	204	163	132	108	90	76	72	61	V
12 x .075	2837	1682	1082	738	527	390	297	232	185	149	123	102	86	73	63	V

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Deflection Capacities Based on Cracked Creep Transformed Properties, [1/240] for Total Load Deflection

Section Properties of Lost Form System [2.5" Topping]

s= 16 inches

Service Loads Joists Are Capable of Supporting for Different Spans

[psf]

Span [ft]	10	12	14	16	18	20	22	24	26	28	30	32	34	36	38	40
5.5 x .048	392	234	152	104	75	56	43	38	V	V	V	V	V	V	V	V
5.5 x .060	468	280	182	125	89	67	58	44	35	V	V	V	V	V	V	V
5.5 x .075	529	316	204	140	101	75	57	45	36	V	V	V	V	V	V	V
6 x .036	347	208	135	93	66	49	38	33	V	V	V	V	V	V	V	V
6 x .048	436	261	169	116	83	62	47	42	33	V	V	V	V	V	V	V
6 x .060	519	310	201	138	99	74	64	49	39	V	V	V	V	V	V	V
6 x .075	587	351	227	156	112	83	63	50	40	36	V	V	V	V	V	V
7.25 x .048	633	378	244	168	120	89	68	53	43	39	V	V	V	V	V	V
7.25 x .060	757	452	292	200	144	107	81	64	57	46	37	V	V	V	V	V
7.25 x .075	859	512	330	226	162	120	92	72	57	46	38	V	V	V	V	V
8 x .048	731	436	282	193	139	103	79	62	49	45	37	V	V	V	V	V
8 x .060	868	518	335	229	164	122	93	73	58	53	43	V	V	V	V	V
8 x .075	992	591	382	261	187	139	106	83	66	64	44	37	V	V	V	V
9.25 x .060	1200	715	461	316	226	168	128	100	80	65	53	49	41	V	V	V
9.25 x .075	1378	820	529	361	259	192	146	114	91	74	61	51	43	V	V	V
10 x .060	1353	806	520	356	255	189	144	113	90	73	60	56	46	V	V	V
10 x .075	1544	918	592	405	290	215	164	128	102	83	68	57	48	41	V	V
12 x .060	1977	1176	758	518	371	275	210	164	130	106	87	72	61	58	49	V
12 x .075	2270	1348	868	593	424	314	239	187	149	120	99	82	69	59	51	44

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Section Properties of Lost Form System [2.5" Topping]

s= 20 inches

Service Loads Joists Are Capable of Supporting for Different Spans

[psf]

Span [ft]	10	12	14	16	18	20	22	24	26	28	30	32	34	36	38	40
5.5 x .048	327	196	127	87	63	47	36	32	25	V	V	V	V	V	V	V
5.5 x .060	393	235	152	105	75	56	49	37	29	V	V	V	V	V	V	V
5.5 x .075	445	266	172	118	85	63	48	38	30	28	V	V	V	V	V	V
6 x .036	288	173	112	77	55	41	31	28	V	V	V	V	V	V	V	V
6 x .048	364	218	141	97	70	52	40	35	28	V	V	V	V	V	V	V
6 x .060	435	260	169	116	83	62	54	41	33	V	V	V	V	V	V	V
6 x .075	493	295	191	131	94	70	53	42	33	31	V	V	V	V	V	V
7.25 x .048	527	315	204	140	100	75	57	45	36	32	V	V	V	V	V	V
7.25 x .060	633	378	245	168	120	89	68	53	48	39	31	V	V	V	V	V
7.25 x .075	720	429	277	190	136	101	77	60	48	39	32	27	V	V	V	V
8 x .048	608	363	235	161	116	86	66	51	41	38	31	V	V	V	V	V
8 x .060	725	433	280	192	138	102	78	61	49	44	36	V	V	V	V	V
8 x .075	831	495	320	219	157	117	89	70	56	45	37	31	V	V	V	V
9.25 x .060	1001	597	385	264	189	140	107	84	67	54	45	41	35	V	V	V
9.25 x .075	1154	687	443	303	217	161	123	96	77	62	51	42	36	30	V	V
10 x .060	1129	673	435	298	213	158	121	94	75	61	50	47	39	V	V	V
10 x .075	1293	769	496	340	243	180	138	107	86	69	57	48	40	34	V	V
12 x .060	1650	982	633	433	310	230	175	137	109	89	73	61	51	49	41	V
12 x .075	1901	1130	728	498	356	264	201	157	125	101	83	69	58	50	43	37

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Deflection Capacities Based on Cracked Creep Transformed Properties, [l/240] for Total Load Deflection

6.2 GEMINI LOST FORM BEAM LOAD TABLES

6.2.1 Beam Load Table Specifications

1.0 Load Table Calculations

.1 These tables were completed following accepted engineering practice and experimental results.

1.1 The effective width used is in accordance with clause 17.3.2.1 of CAN3-S16.1-M78.

1.2 These tables were calculated on the basis that the beams will be shored at 2400 o/c during construction.

1.3 The concrete will be placed to the bottom of the beam cold-formed steel channels.

.2 The ultimate capacity of the beams is limited using the following criteria:

2.1 The moment capacity of the section is based on section 10 of CAN3-A23.3-M84.

2.2 The Shear Capacity is limited using CAN3-S136-M84 clause 6.4.5 with with web stiffeners at a/h = 1.0. or CAN3-A23.3-M84 clause 11.3.4.1.

2.0 Materials

.1 Steel Channels: Cold-formed channels fabricated by Bailey Metals or Mantane Construction Products under licence of Gemini Structural Systems. The Section properties used were taken from the Bailey Mantane product catalogue. The steel yield strength is to be at least 230 MPa. for steel thinner than .06", or 345 MPa for thicker steel.

.2 Concrete : 25 MPa. with a cement: water ratio of 0.50. The concrete is be cured for seven days and is to be shored until the concrete strength achieves 20 MPa.

3.0 Use of the Tables

One set of tables is provided giving ultimate load capacities

.1 The property tables give the section properties from which the load capacities are calculated.

I Moment of Inertia of Composite Section

Mr Moment Capacity of Composite Section

Vr Shear Capacity of Composite Section.

.2 Ultimate Load Table

2.1 The load capacities provided in the tables are the factored capacities of the beam. When checking capacities include the self-weight of the system.

6.2.2 Sample Calculation of Composite Floor Beam Properties

Use beam with 65 mm topping $As = 2 \times 462\ mm^2$
6" x .036" floor joist $Fy = 230$ MPa.
6" x .036" beam channel h & $hj = 152.4$ mm

Moment- $Mr = \varnothing_S A_S F_y [d-a/2]$ A_S- Area of Beam Steel Channels

$$a = \frac{\varnothing_S A_S F_y}{\varnothing c\ .85\ f'c\ b}$$

$b = 16 \times t + 150$ t - Thickness of Concrete Topping

$d = t + 9 + h/2 + hj$ h-height of Beam Steel Channel
hj-height of Joist Channel

$Mr = \varnothing_S A_S F_y [d-a/2]$

$b = 16 \times 65 + 150 = 1190$ mm
$d = 65 + 9 + 152.4 + 152.4/2 = 302.6$ mm

$Mr = 29.94$ kN m

Shear- $Vr = \varnothing c\ .2 \sqrt{f'c}\ b \times .8 \times [t + 9 + h + hj]$
or = Vr from beam channel shear capacity with web stiffeners at $a/h = 1.0$

= 11.53 kN Steel Joist Shear capacity
= **27.71** kN plain concrete strength capacity

Beam Load Tables

Table 6.3

Section Properties of Lost Form System Beam

	Imperial Properties						Metric Properties (for two Channels) Es= 203000 MPa					Composite Properties f'c = 25 148.164				
Channel	Thickness	Depth	Area	I	Vr	Mr	Depth	Area	I	Vr	Mr	y	Area	It	Mr	Vr
	in.	in.	in.^2	in.^4	Kips	in Kips	mm	mm^2	mm^4	kN	kNm	mm	mm^2	mm^4	kNm	kN
5.5 x .048	0.048	5.5	0.473	2.14	2.53	22.9	140	610	1.78E+06	22.51	5.17	258	15689	1.60E+11	36.9	29.7
5.5 x .060	0.06	5.5	0.588	2.65	4.97	42.8	140	759	2.21E+06	44.21	9.67	256	15837	1.55E+11	67.9	58.4
5.5 x .075	0.075	5.5	0.729	3.26	8.79	52.7	140	941	2.71E+06	78.20	11.9	254	16019	1.49E+11	83.6	103.2
6 x .036	0.036	6	0.358	1.83	0.98	18.1	152	462	1.52E+06	8.72	4.09	268	15778	1.87E+11	28.6	27.2
6 x .048	0.048	6	0.473	2.44	2.32	24.1	152	610	2.03E+06	20.64	5.45	266	15926	1.81E+11	37.7	27.2
6 x .060	0.06	6	0.585	2.99	4.52	44.8	152	755	2.49E+06	40.21	10.1	264	16070	1.75E+11	69.0	53.1
6 x .075	0.075	6	0.722	3.63	8.64	54.6	152	932	3.02E+06	76.86	12.3	262	16247	1.69E+11	84.7	101.5
7.25 x .048	0.048	7.3	0.557	4.13	1.93	33.5	184	719	3.44E+06	17.17	7.57	286	16627	2.36E+11	46.6	29.5
7.25 x .060	0.06	7.3	0.693	5.11	3.78	62.9	184	894	4.25E+06	33.63	14.2	284	16803	2.28E+11	85.8	44.4
7.25 x .075	0.075	7.3	0.861	6.31	7.41	77.5	184	1111	5.25E+06	65.92	17.5	281	17020	2.18E+11	105.8	87.0
8 x .048	0.048	8	0.569	4.88	1.75	35.6	203	734	4.06E+06	15.57	8.04	298	16999	2.76E+11	49.0	30.9
8 x .060	0.06	8	0.705	6	3.41	67.5	203	910	4.99E+06	30.34	15.3	296	17174	2.67E+11	89.9	40.0
8 x .075	0.075	8	0.872	7.36	6.62	82.6	203	1125	6.13E+06	58.89	18.7	294	17390	2.56E+11	110.4	77.7
9.25 x .060	0.06	9.3	0.813	9.24	2.97	85.6	235	1049	7.69E+06	26.42	19.3	315	17906	3.33E+11	108.4	34.9
9.25 x .075	0.075	9.3	1.01	11.4	5.82	110	235	1303	9.49E+06	51.77	24.9	312	18161	3.18E+11	133.6	68.3
10 x .060	0.06	10	0.825	10.4	2.73	87.3	254	1065	8.66E+06	24.29	19.7	327	18278	3.82E+11	113.1	34.6
10 x .075	0.075	10	1.02	12.8	5.32	115	254	1316	1.07E+07	47.33	26	324	18529	3.66E+11	138.8	62.5
12 x .060	0.06	12	0.945	16.5	2.28	101	305	1219	1.37E+07	20.28	22.8	357	19381	5.23E+11	138.5	38.2
12 x .075	0.075	12	1.17	20.3	4.45	145	305	1510	1.69E+07	39.59	32.8	354	19672	4.99E+11	170.1	52.3

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ALBERTA
December 13/1989

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Ultimate Capacity without Channel Openings

Section Properties of Lost Form System Beam

Factored Load Carried by Joists in kN/m for Different Spans

Channel	2750	3000	3250	3500	3750	4000	4250	4500	4750	5000	5250	5500	5750	6000	6250	6500
5.5 x .048	21.61	19.81	18.28	16.98	15.84	14.85	13.98	13.20	12.51	11.79	10.70	9.75	8.92	8.19	7.55	6.98
5.5 x .060	42.44	38.91	35.91	33.35	31.13	29.18	27.46	25.94	24.06	21.72	19.70	17.95	16.42	15.08	13.90	12.85
5.5 x .075	75.07	68.81	63.32	54.59	47.56	41.80	37.03	33.03	29.64	26.75	24.26	22.11	20.23	18.58	17.12	15.83
6 x .036	19.81	18.16	16.77	15.57	14.53	13.62	12.66	11.30	10.14	9.15	8.30	7.56	6.92	6.35	5.86	5.41
6 x .048	19.81	18.16	16.77	15.57	14.53	13.62	12.82	12.11	11.47	10.90	10.38	9.91	9.11	8.37	7.71	7.13
6 x .060	38.60	35.38	32.66	30.33	28.31	26.54	24.98	23.59	22.35	21.23	20.03	18.25	16.70	15.34	14.13	13.07
6 x .075	73.79	67.64	62.44	55.29	48.16	42.33	37.50	33.45	30.02	27.09	24.57	22.39	20.48	18.81	17.34	16.03
7.25 x .048	21.48	19.69	18.17	16.87	15.75	14.77	13.90	13.12	12.43	11.81	11.25	10.74	10.27	9.84	9.45	8.82
7.25 x .060	32.28	29.59	27.32	25.36	23.67	22.19	20.89	19.73	18.69	17.75	16.91	16.14	15.44	14.80	14.20	13.66
7.25 x .075	63.28	58.01	53.55	49.72	46.41	43.51	40.95	38.67	36.64	33.86	30.71	27.98	25.60	23.51	21.67	20.03
8 x .048	22.47	20.60	19.02	17.66	16.48	15.45	14.54	13.73	13.01	12.36	11.77	11.24	10.75	10.30	9.89	9.28
8 x .060	29.12	26.70	24.64	22.88	21.36	20.02	18.84	17.80	16.86	16.02	15.25	14.56	13.93	13.35	12.81	12.32
8 x .075	56.54	51.82	47.84	44.42	41.46	38.87	36.58	34.55	32.73	31.09	29.61	28.27	26.72	24.54	22.62	20.91
9.25 x .060	25.36	23.25	21.46	19.93	18.60	17.44	16.41	15.50	14.68	13.95	13.29	12.68	12.13	11.63	11.16	10.73
9.25 x .075	49.70	45.56	42.06	39.05	36.45	34.17	32.16	30.37	28.78	27.34	26.04	24.85	23.77	22.78	21.87	21.03
10 x .060	25.13	23.04	21.27	19.75	18.43	17.28	16.26	15.36	14.55	13.82	13.17	12.57	12.02	11.52	11.06	10.63
10 x .075	45.43	41.65	38.44	35.70	33.32	31.24	29.40	27.77	26.30	24.99	23.80	22.72	21.73	20.82	19.99	19.22
12 x .060	27.79	25.48	23.52	21.84	20.38	19.11	17.98	16.99	16.09	15.29	14.56	13.90	13.29	12.74	12.23	11.76
12 x .075	38.00	34.84	32.16	29.86	27.87	26.13	24.59	23.22	22.00	20.90	19.91	19.00	18.18	17.42	16.72	16.08

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Ultimate Capacity without Channel Openings

Section Properties of Lost Form System Beam

Factored Load Carried by Joists in lbs/ft for Different Spans

Span (ft)	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23
5.5 x .048	1670	1484	1336	1214	1113	1028	954	891	835	752	671	602	544	493	449	411
5.5 x .060	3280	2916	2624	2386	2187	2019	1874	1749	1564	1386	1236	1109	1001	908	827	757
5.5 x .075	5801	5157	4641	4077	3426	2919	2517	2192	1927	1707	1522	1366	1233	1119	1019	932
6 x .036	1531	1361	1225	1114	1021	942	861	750	659	584	521	467	422	383	349	319
6 x .048	1531	1361	1225	1114	1021	942	875	817	766	721	681	615	555	504	459	420
6 x .060	2983	2652	2387	2170	1989	1836	1705	1591	1492	1404	1257	1128	1018	923	841	770
6 x .075	5702	5069	4562	4128	3469	2956	2549	2220	1951	1729	1542	1384	1249	1133	1032	944
7.25 x .048	1660	1475	1328	1207	1107	1021	948	885	830	781	738	699	664	623	568	520
7.25 x .060	2495	2218	1996	1814	1663	1535	1426	1331	1247	1174	1109	1050	998	950	907	868
7.25 x .075	4891	4347	3912	3557	3260	3010	2796	2608	2439	2160	1927	1729	1561	1416	1290	1180
8 x .048	1737	1544	1389	1263	1158	1069	992	926	868	817	772	731	695	656	598	547
8 x .060	2251	2001	1800	1637	1500	1385	1286	1200	1125	1059	1000	948	900	857	818	783
8 x .075	4369	3884	3495	3178	2913	2689	2497	2330	2185	2056	1942	1805	1629	1478	1346	1232
9.25 x .060	1960	1742	1568	1426	1307	1206	1120	1045	980	922	871	825	784	747	713	682
9.25 x .075	3841	3414	3073	2794	2561	2364	2195	2049	1921	1808	1707	1617	1536	1463	1397	1336
10 x .060	1942	1727	1554	1413	1295	1195	1110	1036	971	914	863	818	777	740	706	676
10 x .075	3511	3121	2809	2554	2341	2161	2006	1873	1756	1652	1561	1478	1404	1338	1277	1221
12 x .060	2148	1909	1718	1562	1432	1322	1227	1146	1074	1011	955	904	859	818	781	747
12 x .075	2937	2611	2350	2136	1958	1807	1678	1566	1469	1382	1305	1237	1175	1119	1068	1022

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Ultimate Capacity without Channel Openings

6.3 GEMINI COLUMN LOAD TABLES

6.3.1 Column Load Table Specifications

1.0 Load Table Calculations

.2 The ultimate capacity of the joists was limited using the following criteria:

2.1 The axial capacity of the section was based on section 10 of CAN3-A23.3-M84.

2.0 Materials

.1 Steel Channels: Cold-formed channels fabricated by Bailey Metals or Mantane Construction Products under licence of Gemini Structural Systems. The section properties used were taken from the Bailey Mantane product catalogue. The steel yield strength is to be at least 230 MPa. for steel thinner than .06", or 345 MPa for thicker steel.

.2 Concrete : 25 MPa. with a cement:water ratio of 0.50.

3.0 Use of the Tables

One set of tables is provided giving ultimate load capacities

.1 The property tables give the section properties from which the load capacities are calculated.

Pr Axial Capacity of Composite Section

Lmin Minimum length of Composite Section to obtain full axial capacity.

.2 Ultimate Load Table

2.1 The load capacities provided in the tables are the factored capacities of the column. When checking capacities include the self-weight of the column.

6.3.2 Sample Calculation of Composite Column Properties

Use 150 mm x 300 mm column with two 6" x .036" cold-formed channels

$As = 462\ mm^2$ $Fy = 230$ MPa.

$f'c = 25$ MPa

Limiting Criteria

Axial Capacity- $Pr = Ø_c A_c .85 f'_c + Ø_s A_s F_y$

Ac- Area of Concrete

As- Area of Steel

Channels

Moment Capacity-   This was outside the the scope of this experimental program but work has been done on this subject (see paper by George Abdel-Sayed and Kwok-Cheung Chung) in the Canadian Journal of Civil Engineering volume 14, 1987.

Minimum Length-   Based on experimental results

$$L_{min} = \frac{Ø_s A_s F_y}{Ø_c\ 265\ \text{N/mm}}$$

$$\begin{aligned} Pr &= .6 \times (150 \times 300 - 2 \times 231) \times .85 \times 25 + .9 \times 2 \times 231 \times 230 \\ &= 664\ \text{kN} \end{aligned}$$

$$\begin{aligned} L_{min} &= \frac{.9 \times 2 \times 231 \times 230}{.6 \times 265} \\ &= 602\ \text{mm} \end{aligned}$$

Column Axial Factored Capacity Tables
Table 6.4

Metric

Steel Channel	As [mm^2]	Lmin [mm]	Pr [kN] 150 x 150	150 x 300	150 x 450
6" x .036"	462	600	377	664	950
6" x .048"	610	800	405	692	979
6" x .060"	755	1475	512	799	1085
6" x .075"	932	1820	564	851	1138

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P. THOMSON
David P. Thomson
December 15, 1989

Imperial

Steel Channel	As [in^2]	Lmin [in]	Pr [lbs.] 6" x 6"	6" x 12"	6" x 18"
6" x .036"	0.716	24	86,530	153,120	219,700
6" x .048"	0.946	31	92,920	159,510	226,090
6" x .060"	1.17	58	117,080	183,670	250,260
6" x .075"	1.45	72	128,920	195,510	262,100

7.0 CONCLUSIONS

7.1 ECONOMIC VIABILITY

A cost analysis indicated that the Gemini II System is economically feasible in residential construction. The system is more suited to larger projects such as apartments, but within a certain economic limit it can, be used in detached housing as well. The cost analysis showed that the Gemini II System was 13 % more economical for a seven storey apartment building than the most economical conventional method of construction, but was 12 % more expensive than conventional wood framing in a detached split-level house. The Gemini System is roughly the same cost as a manufactured composite joist floor system for wood framing. Although this study focussed on residential construction, the results of the cost analysis indicate that the Gemini System II is worth considering as an alternative in both commercial and industrial projects. Gemini System II is both versitile and structurally sound, and it should find wide application in the construction industry.

7.2 EXPERIMENTAL CONCLUSIONS

The test program verified load deflection behaviour of the floor joist, beam, and the column components. The program indicated that the floor joists can be designed as composite members using the section modulus of the transformed cracked moment of inertia to limit the moment capacity, and the steel channel to limit the shear capacity.

The floor beams were proven to behave compositely. The moment capacity of the beams can be calculated as recommended in the reinforced concrete code CAN3-A23.3, while the shear is limited by shear strength of the stiffened steel channels, or the strength of the plain concrete in the beam.

Also, three conclusions could be made about the columns; they behave compositely, the screwed enclosed section has the most ductile behaviour, and the columns can be designed using the combined concrete and steel channels axial strengths.

Further research is recommended in two areas:

The shear capacity of the floor beam is not fully understood, therefore, it is recommended that further testing be completed to verify the extent that the concrete in the beam acts as web stiffeners. Further testing may permit an increase in the shear strength of the beams used in the load tables.

The second area requiring further testing is in verification of the moment capacity models of the composite columns

7.3 LOAD TABLES

Load tables were developed in this study for use by architects and engineers. The tables will permit quick and easy selection of proper steel cold-formed channel sizes for different loads and spans. The specifications provided with the load tables will permit architects or engineers to develop suitable contract specifications. The specifications, together with the load tables, outline all the design criteria, enabling structural engineers to design for unique situations.

APPENDIX A

FLOOR JOIST TEST RESULTS AND ANALYSIS

COLD-FORMED STEEL PROPERTIES

Tension tests were carried out as specified by the Canadian Standards Association on the 18 gauge channels used in the full scale tests of the floor joists. Yield and ultimate strengths of the steel were found but the modulus of elasticity was not and the published value used in S-136 was used. Three 18 gauge coupons were tested by Hardy BBT on June 15, 1988. Two of the coupons were cut in the web near the flanges while the third was removed from the centre of the web. Weighted averages of the yield and ultimate strength were calculated following the procedure recommended by S–136 in chapter 9 as shown below.

$$F_y = \frac{52 \times [F_{y1} + F_{y3}] + 152 \times F_{y2}}{52 + 52 + 152}$$

The weighted average yield strength of the 18 gauge steel was 316.5 MPa and the ultimate strength was 361.6 MPa. The results indicate that the steel meet the requirements of S-136 with the 38% elongation (the minimum required elongation being 10%) and F_u / F_y of 1.14 versus the required minimum of 1.08. Copies of the results of the three coupon tests are included on the next page.

Hardy BBT Limited
CONSULTING ENGINEERING & PROFESSIONAL SERVICES

METAL TEST REPORT

To: Gemini Structrual Systems
32 Castleglen Ct. N.E.
Calgary, Alberta
T3J 2B8

Lab. Order No. CA-08788
Type of Sample
Project
Source
Sampled by
Date Sampled
Date Received
Date Tested November 25, 1988
Date Reported
Laboratory

Copies to: Campbell Woodall Associates
Attn: Mr. Dave Thompson

A. Tension Tests

Sample Mark	1	2
Size	.500 x .051 .349 x .030	.499 x .051 .336 x .029
Init. Area-sq. ins.	0.0255	0.0254
Final Area-sq. ins.	0.0105	0.0097
Total Load-lbs.	1280	1268
Ult. Stress-psi.	50,200	49,900
Yield Load-lbs.	1060	1040
Yield Stress-psi.	41,600	40,900
Init. Gage-ins.	2.000	2.000
Final Gage-ins.	2.792	2.802
Elongation-percent	39.6	40.1
Red. in Area-percent	58.8	61.8
Type of Failure		
Fracture — Modulus of Elasticity 10^3 psi	39,216	46,258

B. Bend Tests

Sample Mark

Passed-Failed

C. Other Tests

D. Remarks

Certified: A Green

HT 22 - 79/06

Hardy BBT Limited
CONSULTING ENGINEERING & PROFESSIONAL SERVICES

Lab. Order No. CA-08643
Type of Sample Tensile
Project
Source Client
Sampled by
Date Sampled
Date Received
Date Tested June 15, 1988
Date Reported
Laboratory
Copies to: Mr. D. P. Thomspn, P. Eng.

METAL TEST REPORT

Campbell Woodall & Associates
Consulting Engineers Ltd.
250, 1210 - 8 Street S.W.
Calgary, Alberta
T2R 1L3

Tension Tests — On samples from 18 Gauge Channel

Sample Mark	1	2	3
Size	12.56 x 1.27 9.2 x .9	12.41 x 1.27 9.35 x .9	12.51 x 1.27 10.0 x .95
Init. Area-sq. mm	15.95	15.76	15.89
Final Area-sq. mm	8.28	8.41	9.5
Total Load-N	5 835	5 693	5 202
Ult. Stress-MPa	365.8	361.2	358.8
Yield Load-N	5 248	4 892	5 115
Yield Stress-MPa	329.0	310.4	321.9
Init. Gage- mm	50.0	50.0	50.0
Final Gage-mm	70.0	69.0	68.0
Elongation-percent	40.0	38.0	36.0
Red. in Area-percent	48.1	46.6	40.2
Type of Failure			
Fracture			

Bend Tests

Sample Mark

Passed-Failed

Other Tests

Remarks

Samples 1 and 3 from edges of Channel Web
Sample 2 from Centre of Web

Certified: [signature]

HANSON
MATERIALS ENGINEERING
#4, 3650 - 21 Street N.E., Calgary, Alberta T2E 6V6
Tel. (403)291-3126 (24 hrs.) Telex 037-3755 Fax 250-1015

TENSILE & GUIDED BEND TEST REPORT

CLIENT: Campbell, Woodall & Associates PO #:

ADDRESS: 250, 1210 - 5 St. SW, Calgary DATE: December 12/88

ATTENTION: Mr. D. Thompson HME #: C88-12-765

MATERIAL DESCRIPTION: Cold Formed Channel

SPECIMEN TYPE: Longitudinal Reduced Section Tensiles

TECHNICIAN: D. Haigh

	SAMPLE A		SAMPLE B	
	Metric	Imperial	Metric	Imperial
Dimensions:				
-Width	5.72 mm	0.225 in	6.15 mm	0.242 in
-Thickness	1.27 mm	0.050 in	1.27 mm	0.050 in
-Area	7.26 mm^2	0.011 in^2	7.81 mm^2	0.012 in^2
Yield Load	2.24 kN	504.1 lbs	2.31 kN	520 lbs
Yield Strength	309 MPa	44,809 psi	296 MPa	42,975 psi
Ultimate Load	2.64 kN	594.1 lbs	2.80 kN	628.1 lbs
Ultimate Strength	364 MPa	52,809 psi	358 MPa	51,909 psi
Elongation	37%	37%	32%	32%

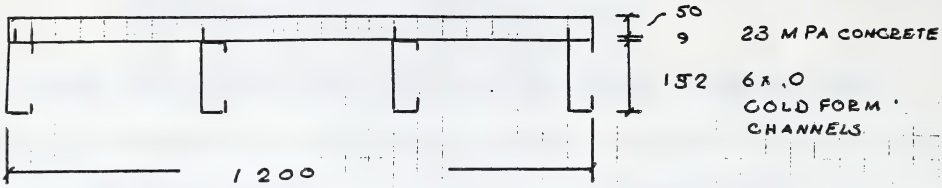
	Sample A		Sample B	
Modulus of Elasticity	166 GPa	24.14×10^6 psi	216 GPa	31.43×10^6 psi

The average modulus of elasticity is: 191 GPa (27.70×10^6 psi). This value is in the modulus range for steel 193 - 206 GPa (28×10^6 psi - 30×10^6 psi)

HANSON MATERIALS ENGINEERING

Load Deflection Readings from the Floor Panel Load Tests

Floor Panel I				Floor Panel II			
Load [kN]	Δ_q [mm]	Δ_m [mm]	Δ_q [mm]	Load [kN]	Δ_q [mm]	Δ_m [mm]	Δ_q [mm]
0	2	3	2	0	0	0	0
5.5	3	4.5	3.5	19.6	3.5	6.5	4.5
11	4	6	6	39.2	11.5	17.5	12.5
16.5	5	8	6.5	39.2	11	17.5	12
22	6.5	9	7.5	0	4.5	7	5.5
25.5	7	11	8.5	19.6	7.5	13	9
25.5	7.5	12	9	39.2	11.5	18	13
0	2	3.5	3.5	0	4.5	7	5.5
31.4	9.5	14	10.5	19.6	8	12.5	8
36.9	10	16	11.5	39.2	11.5	17.5	12.5
38.8	11.5	16.5	12.5	0	4.5	7	5.5
38.8	10.5	16.5	12.5	19.6	6.5	14	9
49.4	14	20.5	14.5	39.2	11.5	18.5	12.5
60.4	18	28.5	20	47.1	20	28	21
60.4	23	36	26	58.8	26	40	26.5
0	9	17.5	10.5	-	-	-	-



CRACKED TRANSFORMED SECTION PROPERTIES

AMOUNT OF CONCRETE IN COMPRESSION

$$a = \left[\left(\frac{2db}{nAs} + 1 \right)^{1/2} - 1 \right] \frac{nAs}{b}$$

$n = \frac{Es}{Ec}$ $\quad Es = 191$ GPa $\quad$ FROM TESTS

$\quad$ $Ec = 5000\sqrt{23}$ $\quad fc = 23$ MPa

$= 7.97$ $\quad = 23.98$ GPa

b = 1200 mm
d = 50 + 9 + 152/2 = 135 mm
As = 1178

a = 37.5 mm $\quad$ yb = 152.4 + 50 + 9 − 37.5
$\quad$ = 173.9 mm

	y	A	y'	$\frac{Ay'^2}{10^3}$	$\frac{I}{10^3}$	$\frac{I + Ay'^2}{10^3}$
CHANNEL	76.2	1178.1	97.7	11,254.9	3,918.5	15,173.4
CONCRETE	192.7	5643.1	18.7	1,979.5	659.8	2,639.4
Σ		6821.4		13,234.5	4,578.3	17,812.8

$I_T = 17.812 \times 10^6 \text{ mm}^4$

SHEAR CONNECTION $\quad Q_R \geq As\,fy \quad As = 295$
$\quad Fy = 316.5$

$Q_R = 3.4 \times \frac{295}{231} \times \frac{316.5}{290.2} = 4.74$ kN/TAB

$$M_{req} = \frac{As\,Fy}{Q_R/TAB}$$

= 20 TABS

∴ THERE IS 60% SHEAR CONNECTION

$$I_e = I_s + .85\,e^{.25}(I_T - I_s)$$

$= 15.380 \times 10^6 \text{ mm}^4$

$$\Delta = \frac{14}{1000}\frac{WL^3}{EI}$$

$= .234\ W$

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PROJECT: GEMINI FLOOR JOIST PANEL I	DATE:
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DEFLECTION COMPARISON

Δ TEST $= \frac{6-3}{11} = .273$ W   FOR 1ST LOADING CYCLE

$\frac{\Delta \text{ TEST}}{\Delta \text{ CAL}} = 1.17$

Δ TEST $= \frac{16-3.5}{36.9} = .339$ W   FOR 2ND LOADING CYCLE

$\frac{\Delta \text{ TEST}}{\Delta \text{ CAL}} = 1.45$

MOMENT CAPACITY

$M_E = F_y \, S_b$ $S_b = \frac{I_e}{y_b}$ $F_y = 316.5$ MPa

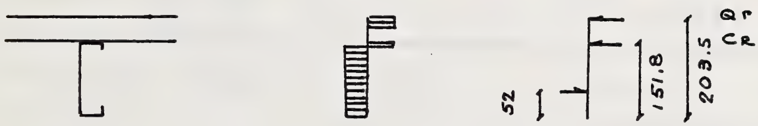
$= 27.98$ kNm

$W_E = 50.97 - 5.16$ kN $W_{TEST} = 60.4$ kN

$= 45.81$ kN

$\frac{W_{TEST}}{W_{CAL}} = 1.31$

ULTIMATE MOMENT CAPACITY



$C_R = \frac{A_s F_y - Q_R}{2}$ $Q_R = .6 \, F_y A_s$

$= .2 \, A_s F_y$

$C_R = 18.67$ kN $\times 4$ $F_y = 316.5$
$Q_R = 56.02$ kN $\times 4$ $A_s = 4 \times 295$

$M_R = Q_R \, (203.5 - 52) + C_R \, (151.8 - 52)$

$= 41.4$ kN m

$W_U = 75.42$ kN $- 5.16$

$= 70.26$

$\frac{W_{TEST}}{W_U} = .86$

$\Delta_{MID} = \frac{4 \, \varphi_{MID} \, L^2}{48}$ φ_{MID} CURVATURE AT MID SPAN

$= 53.4$ $\varphi_{MID} = \frac{.003}{211.4 - 148.7}$
$= 4.786 \times 10^{-5}$

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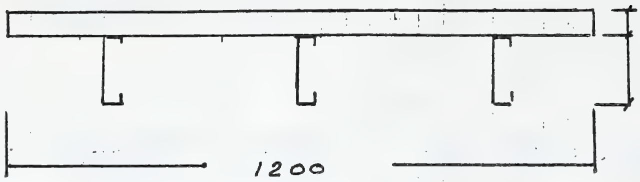

CRACKED TRANSFORMED SECTION PROPERTIES

AMOUNT OF CONCRETE IN COMPRESSION

$$a = \left[\left(\frac{2db}{nA_s} + 1 \right)^{1/2} - 1 \right] \frac{nA_s}{b}$$

$n = 7.97$
$E_s = 191$ GPa
$E_c = 24$ GPa

$= 34.38$ mm

$y_b = 197.0$ mm

	y	A	y'	Ay'^2	I	$I + Ay'^2$
CHANNEL	76.2	885	100.8	8980.9	2938.9	11919.7
CONCRETE	194.2	5179.8	17.2	1530.8	510.3	2041.1
		6063.4		10,511.7	3449.1	13960.8

$I_T = 13960.8 \times 10^3$ mm^4

SHEAR CONNECTION 60 % SEE JOIST-1

$I_e = I_s + .85\, e^{.25} (I_T - I_s)$

$= 11.184 \times 10^6$ mm^4

$\Delta = \frac{14}{1000} \frac{WL^3}{EI}$

$= .322$ W

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PROJECT: GEMINI FLOOR JOIST PANEL II	DATE:
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Deflection Comparison

Δ TEST = $\frac{6.5 - 0}{19.6}$ = .332 FOR 1[ST] LOAD CYCLE

$\frac{\Delta \text{ TEST}}{\Delta \text{ CAL}}$ = 1.03

Δ TEST = $\frac{17.5 - 7}{39.2}$ = .268 FOR 2[ND] LOAD CYCLE

$\frac{\Delta \text{ TEST}}{\Delta \text{ CAL}}$ = .83

Δ TEST = $\frac{18.5 - 7}{39.2}$ = .293 FOR 3[RD] LOAD CYCLE

$\frac{\Delta \text{ TEST}}{\Delta \text{ CAL}}$ = .91

Moment Capacity

$M_E = F_y S_b$

$= 20.0$ kNm

$W_E = 36.41 - 5.16$

$= 31.24$

$W_{TEST} = 58.8$ kN

$\frac{W_{TEST}}{W_{CAL}} = 1.88$

$M_U = Q_R(203.5 - 52) + C_R(151.8 - 52)$ FROM JOIST-2

$= 31.05$ kNm

$W_U = 56.56 - 5.16$

$= 51.4$ kN

$\frac{W_{TEST}}{W_{CAL}} = 1.14$

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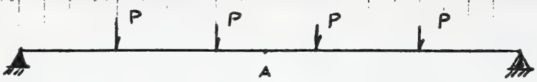
DEFLECTIONS



$W = 4P$

$$\Delta_A = \Sigma \frac{P b_i}{12EI}\left(\frac{3}{4}L^2 - b_i^2\right) \qquad b = \frac{L}{5}, \frac{2L}{5}$$

$$= 2\times\left[\frac{P}{12EI}\left(\frac{L}{5}\left(\frac{3}{4}L^2 - \frac{L^2}{25}\right) + \frac{2}{5}L\left(\frac{3}{4}L^2 - \frac{4L^2}{25}\right)\right)\right]$$

$$= 2\times\left[\frac{P}{60EI}\left[\frac{L^3}{2} + \frac{2L^3}{100}(75 - 16)\right]\right]$$

$$= 2\times\left[\frac{PL^3}{6000EI} \times 168\right]$$

$$= \frac{14}{1000}\frac{WL^3}{EI}$$

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PROJECT: GEMINI FLOOR JOIST DATE:
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SOURCES OF ERRORS

REDUNDANCY OF END REACTIONS

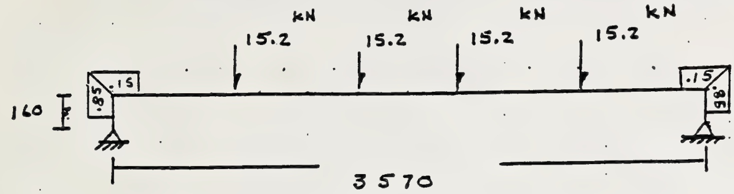
REDUCTION OF SPAN

$L_o = 3660 - 90 = 3570$
$L_i = L_o - 90$
$= 3480$ mm

ERROR ON MOMENT $= [L_i/L_o]^2 = .95$ 1.05 $M = \omega L^2/8$

" " DEFLECTION $= [L_i/L_o]^4 = .90$ 1.11 $\Delta = \frac{5}{384}\frac{\omega L^4}{EI}$

END THRUST



$k_1 = \frac{EI}{L} = \frac{\overset{12.65}{\cancel{12.69}} \times 10^6}{3570} = 3543 \text{ mm}^3$

$k_2 = \frac{EI}{L} = \frac{4.46 \times 10^6}{160} \frac{3}{4} = 20906 \text{ mm}^3$

$M_{END} = \Sigma \; \frac{Pa^2b}{L^2} + \frac{Pab^2}{L^2}$
$= 13.35$ kN m

$M_{END} = .85 \times M$
$= 11.35$ kNm
$= .188 \; W$

MAX END MOMENT BEFORE CONCRETE CRACKS

$M_{MAX} = S_{TOP} \; fr \; n$ $fr = .6\sqrt{fc}\; n$
$= 22.9$ MPa

$S_{TOP} = 410{,}133 \text{ mm}^3$
$M_{MAX} = 9.39$ kN m   4 - JOISTS
12.09 kNm   CAPACITY FROM JOIST

$\therefore \; M_{END} = 11.35$ kNm

$\therefore \; M_{MID} = M_o - M_{END}$

$= \frac{12}{5} WL - \frac{1.3}{5} WL$

$\frac{M_{MID}}{M_o} = \frac{.3}{12} = .025$

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PROJECT: GEMINI FLOOR JOIST	DATE:
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APPENDIX B

FLOOR BEAM TEST RESULTS AND ANALYSIS

Load Deflection Readings from the Floor Beam Load Test

Load [kN]	Δ [mm]	Load [kN]	Δ [mm]
0	1.5	0	4.5
5.5	3.5	54.9	10.5
11	2	62.8	10
16.5	2.5	74.5	8
22	5	74.5	8
27.5	6.5	81.6	10.5
27.5	8.5	0	2.5
0	3.5	0	1
27.5	6.5	27.6	4.5
38.8	10	53.4	5.5
48.6	11	81.1	9.5
48.6	10	96.8	#######

REPORT ON INSTALLATION AND READING OF STRAIN GAUGES FOR LOAD/DEFLECTION TESTING PROGRAM ON HERB SHULGER COMPOSITE CONCRETE BEAM AND FLOOR PANEL

Submitted to:

CAMPBELL WOODALL & ASSOCIATES

CONSULTING ENGINEERS LTD.

Prepared by:

CURTIS ENGINEERING ASSOCIATES LTD.
908 D - 53rd Avenue N.E., Calgary
Telephone 295-0947   Fax 274-7359

July, 1989
File: 289-106-7

1.0 INTRODUCTION

Under authorization from Campbell Woodall & Associates Consulting Engineers Ltd., a series of thirty (30) strain gauges were installed at representative locations on the above noted Composite Concrete Beam and Floor Panel to measure the insitu strains during load testing. All strain gauge locations were selected and detailed at the test site by Mr. Dave Thompson of Campbell Woodall & Associates Consulting Engineers Ltd. A schematic plan of test gauge locations is attached herewith, ref. Plate I-1.

Strain gauges utilized on the testing project were type CEA-06-125UW-120 strain gauges. A GIT 1300 gauge installation tester and a P350 Strain Indicator were used for installing and taking strain gauge measurements under load conditions.

A series of four (4) loadings were applied to the beam configurations to achieve failure conditions. The beam deflection, under various load conditions, was measured at 1/4 and 1/2 point of the beam span by means of deflection gauges and survey instrument.

Load testing of the subject beam sections was carried out on July 5, 1989, at the Rocky Mountain pre cast plant, Calgary, Alberta.

Test results of the above exercise are detailed below.

Strain is measured in micro inches per inch and expressed as a percentage (%).

Yours very truly,

CURTIS ENGINEERING ASSOCIATES LTD.

W. E. Curtis, M.Sc., P.Eng.
General Manager

WEC

STRAIN GAUGE NO.	STRAIN GAUGE READING AT ZERO LOAD	STRAIN GAUGE READING UNDER LOAD 1	STRAIN UNDER LOAD 1	STRAIN GAUGE READING UNDER LOAD 2	STRAIN UNDER LOAD 2	STRAIN GAUGE READING UNDER LOAD 3	STRAIN UNDER LOAD 3	STRAIN GAUGE READING UNDER LOAD 4	STRAIN UNDER LOAD 4
1	(-) 1036	(-) 1054	(-) 1.8%	(Faulty)	(Faulty)	(-) 1070	(-) 3.4%	(-) 1060	(-) 2.4%
2	(-) 2424	(-) 2526	(-)10.2%	(-) 2554	(-)13.0%	(-) 2674	(-)25.0%	(-) 2630	(-)20.6%
3	(+) 64	(+) 66	(+) 0.2%	(+) 60	(+) 0.4%	(+) 90	(+) 2.6%	(+) 115	(+) 5.1%
4	(-) 880	(-) 860	(-) 2.0%	(-) 872	(-) 0.8%	(-) 883	(-) 0.3%	(-) 884	(-) 0.4%
5	(+) 590	(+) 602	(+) 1.2%	(+) 636	(+) 4.6%	(+) 627	(+) 3.7%	(+) 602	(+) 1.2%
6	(-) 4574	(-) 4576	(-) 0.2%	(-) 4606	(-) 3.2%	(-) 4592	(-) 1.8%	(-) 4584	(-) 1.0%
7	(+) 794	(+) 818	(+) 2.4%	(+) 850	(+) 5.6%	(+) 854	(+) 6.0%	(+) 846	(+) 5.2%
8	(+) 488	(+) 586	(+) 9.8%	(+) 964	(+)47.6%	(+) 1143	(+)65.5%	(+) 1888	(+)140.0%
9	(+) 458	(+) 576	(+)11.8%	(+) 1068	(+)61.0%	(+) 1286	(+)82.8%	(+) 1755	(+)129.7%
10	(+) 464	(+) 500	(+) 3.6%	(+) 734	(+)27.0%	(+) 970	(+)50.6%	(+) 1198	(+)72.9%
11	(-) 114	(-) 68	(-) 4.6%	(+) 232	(+)34.6%	(+) 574	(+)68.8%	(+) 875	(+)98.9%
12	(+) 914	(+) 924	(+) 1.0%	(+) 922	(+) 0.8%	Broken			
13	(+) 92	(+) 101	(+) 0.9%	(+) 94	(+) 0.2%	(+) 90	(+) 0.2%	(+) 95	(+) 0.3%
14	(+) 355	(+) 373	(+) 1.8%	(+) 652	(+)29.7%	(+) 825	(+)47.0%	(+) 1048	(+)69.3%

STRAIN GAUGE NO.	STRAIN GAUGE READING AT ZERO LOAD	STRAIN GAUGE READING UNDER LOAD 1	STRAIN UNDER LOAD 1	STRAIN GAUGE READING UNDER LOAD 2	STRAIN UNDER LOAD 2	STRAIN GAUGE READING UNDER LOAD 3	STRAIN UNDER LOAD 3	STRAIN GAUGE READING UNDER LOAD 4	STRAIN UNDER LOAD 4
15	(-) 296	(-) 292	(-) 0.4%	(-) 176	(-)12.0%	(-) 80	(-)21.6%	(-) 4	(-)30.0%
16	(-) 1378	(-) 1390	(-) 1.2%	(-) 1410	(-) 3.2%	(-) 1465	(-) 8.7%	(-) 1430	(-) 5.2%
17	(-) 158	(-) 156	(-) 0.2%	(-) 178	(-) 2.0%	(-) 138	(-) 2.0%	(-) 56	(-)21.4%
18	(+) 946	(+) 976	(+) 3.0%	(+) 1002	(+) 5.6%	(+) 1018	(+) 7.2%	(+) 1043	(+) 9.7%
19	(+) 786	(+) 800	(+) 1.4%	(+) 807	(+) 2.1%	(+) 815	(+) 2.9%	(+) 858	(+) 7.2%
20	(-) 568	(-) 548	(-) 2.0%	(-) 578	(-) 1.0%	(-) 564	(-) 0.4%	(-) 492	(-) 7.6%
21	(-) 339	(-) 322	(-) 1.7%	(-) 315	(-) 2.4%	(-) 305	(-) 3.4%	(-) 498	(-)15.9%
22	(-) 2870	(-) 2822	(-) 4.8%	(-) 1788	(-)108.2%	(-) 782	(-)208.8%	(-) 285	(-)258.5%
23	(-) 348	(-) 274	(-) 7.4%	(+) 77	(-)42.5%	(+) 218	(-)56.6%	(+) 1390	(-)173.8%
24	(+) 1014	(+) 1140	(+)12.6%	(+) 1641	(+)62.7%	(+) 1884	(+)87.0%	(+) 2388	(+)137.4%
25	(+) 262	(+) 254	(+) 0.8%	(+) 232	(+) 3.0%	(+) 244	(+) 1.8%	(+) 248	(+) 1.4%
26	(+) 352	(+) 370	(+) 1.8%	(+) 494	(+)14.2%	(+) 764	(+)41.2%	(+) 940	(+)58.8%
27	(-) 1037	(-) 1030	(-) 0.7%	(-) 1035	(-) 0.2%	(-) 1034	(-) 0.3%	(-) 1032	(-) 0.5%
28	Destroyed								

STRAIN GAUGE NO.	STRAIN GAUGE READING AT ZERO LOAD	STRAIN GAUGE READING UNDER LOAD 1	STRAIN UNDER LOAD 1	STRAIN GAUGE READING UNDER LOAD 2	STRAIN UNDER LOAD 2	STRAIN GAUGE READING UNDER LOAD 3	STRAIN UNDER LOAD 3	STRAIN GAUGE READING UNDER LOAD 4	STRAIN UNDER LOAD 4
29	(+) 550	(+) 572	(+) 2.2%	(+) 664	(+)11.4%	(+) 1032	(+)48.2%	(+) 1238	(+)68.8%
30	(-) 790	(-) 855	(-) 6.5%	Broken					

ELEVATIONS ON BEAM TEST POINTS (North to South)

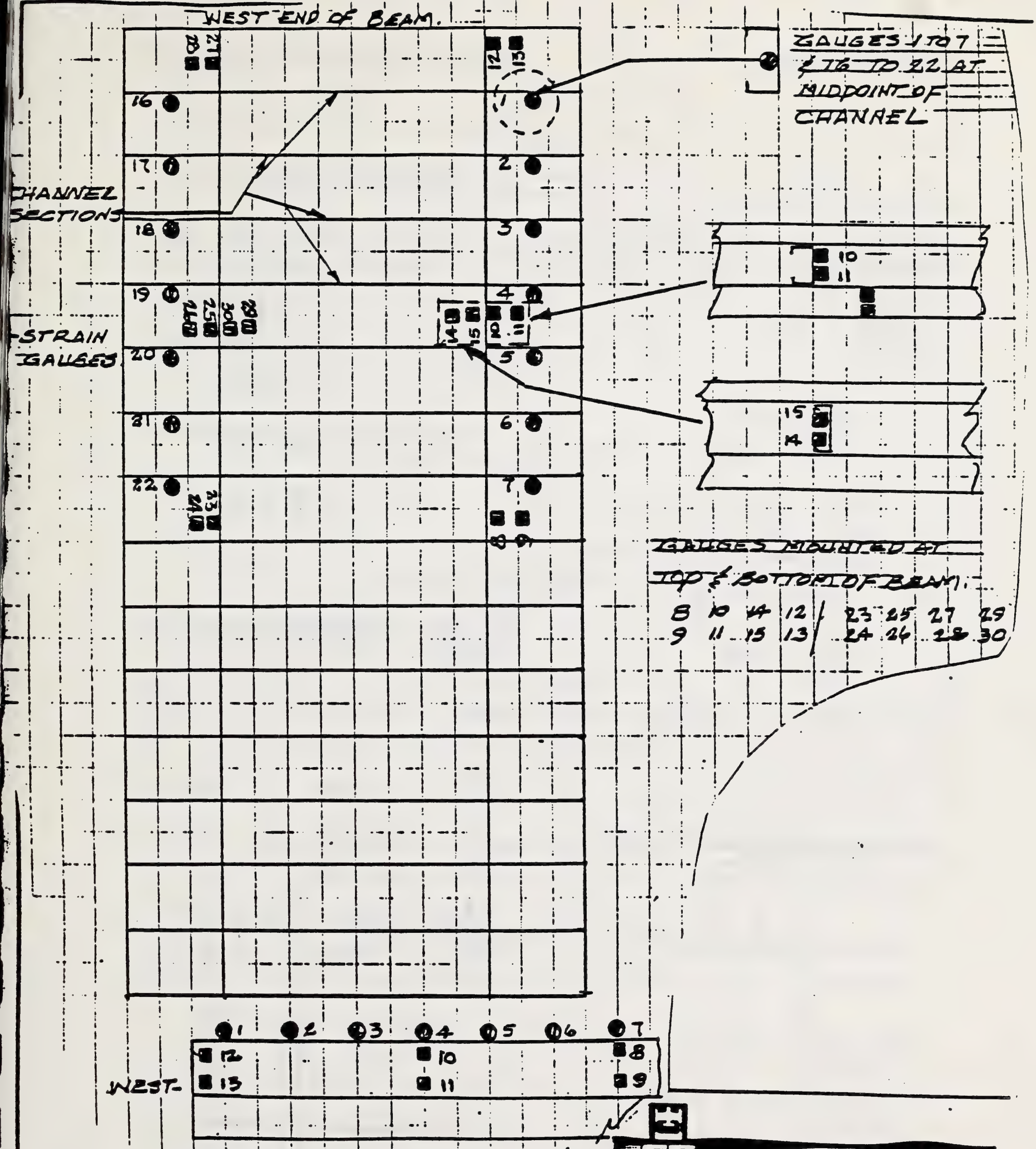
BEAM POINT	LOAD CONDITIONS INITIAL ZERO LOAD	1st LOAD	2nd LOAD	3rd LOAD	4th LOAD
End of Beam	964 mm	970 mm	971 mm	974 mm	978 mm
1/4 Point	961 mm	970 mm	977 mm	983 mm	996 mm
Mid Span	953 mm	960 mm	972 mm	981 mm	995 mm
1/4 Point	955 mm	962 mm	969 mm	975 mm	983 mm
End of Beam	947 mm	951 mm	950 mm	952 mm	953 mm

DEFLECTION DIAL GAUGE READINGS ON BEAM TEST POINTS

(Dial Guage Reading in 1/1000 inch)

BEAM POINT	LOAD CONDITIONS INITIAL ZERO LOAD	1st LOAD	2nd LOAD	3rd LOAD	4th LOAD
1/4 Point	1975	1913	1651 1623*	1535	1436
Mid Span	1790	1623	1339 1325*	1192	92
1/4 Point	565	507	240 210*	1452**	1078

* - After 1/2 hour under same load.
** - Gauge past zero and moving down.



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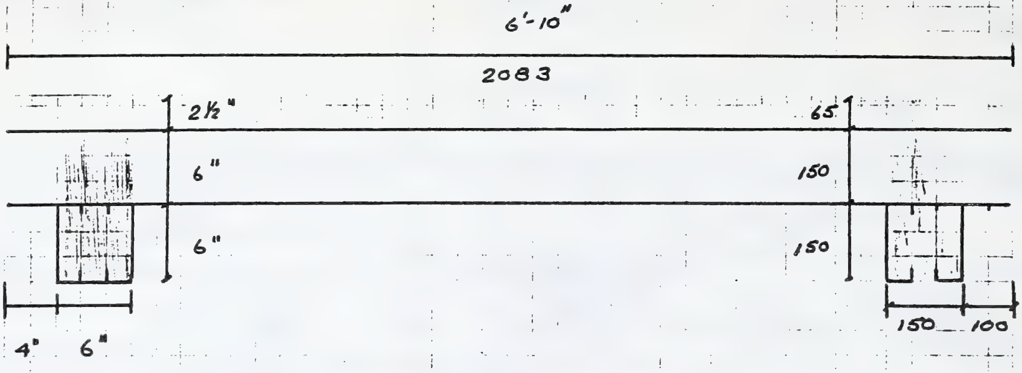
STRAIN GAUGE LOCATIONS

ON TEST BEAM

DRN. BY	B.C.	DATE	July 20/89
SCALE	N.T.S.	DWG.	PLATE I-1

Beam Section



$f'_c = 22.3$ MPa

$V_c = .2\sqrt{f_c}\ b\,d_v \times 2$ $\qquad d_v = .8h \qquad b = 150$

$= .8 \times 365 = 292$ mm

$= 41.4$ kN $\times 2$

$= 82.7$ kN

$= 18.6$ kips

$M_r = A_s F_y (d - \frac{a}{2})$ $\qquad d = 65 + 150 + 75$

$= 290$

$= 86.73$ kN m $\qquad a = \frac{A_s F_y}{b \times .85 f_c} = \frac{1000\ \text{mm}^2 \times 307\ \text{MPa}}{1080\ \text{mm} \times .85 \times 22.3}$

$= 64$ kft

$= 768$ k" $\qquad = 15$ mm

$M_r = A_s F_u (d - \frac{a}{2})$ $\qquad a = 15 \times F_u / F_y =$

=

=

=

Transformed Section

$f'_c = 28.3$ MPa $\qquad E_c = 23.6$ GPa $\qquad n = E_s/E_c = 8.26$

$\omega = 5.1$ kN/m

	y mm	A/n mm²	y' mm	Ay'² 10⁶mm⁴	I 10⁶mm⁴	Ay'² + I 10⁶mm⁴
Channel	76	1000	179	32.00	3.26	35.26
Channel Concrete	152	11190	103	118.42	86.17	204.60
Topping Concrete	336	16395	81	107.90	5.77	113.67
Σ	255	28585	-	258.32	95.21	353.53

$I_T = 353.5 \times 10^6$ mm⁴

$S_b = 1386.4 \times 10^3$ mm³

$M_{cr} = S_b\, n f_r$ $\qquad M_E = S_b f_y$

$= 32.44$ kNm $\qquad = 425.6$ kNm

$= 23.93$ k' $\qquad = 313.9$ k'

$= 287.1$ k" $\qquad = 3767$ k"

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CRACK TRANSFORMED SECTION

$f_c = 22.3$ MPa $\quad n = \frac{E_s}{E_c} = 8.26$
$E_c = 23.6$ GPa

$h = \frac{n A_s}{b} = \frac{8.26 \times 1000}{2083} = 4.0$ mm

$I_{T_{cr}} = 45.6 \times 10^6\ mm^4 \leftarrow \quad 86.8 \times 10^6\ mm^4$

$s_b = 205.9 \times 10^3\ mm^3 \quad - \quad 237.78 \times 10^3\ mm^3$

$M_E = s_b F_y$

$= \cancel{63.21}$ kNm $\quad$ 72.0 kNm
$= \cancel{46.62}$ k' $\quad$ 53.8 k'
$= \cancel{559.5}$ k" $\quad$ 646.1 k"

DEFLECTION CALCULATIONS

$\Delta = \frac{5}{384} \frac{\omega l^4}{EI}$ WHEN FULLY LOADED

$\Delta =$ WHEN THE PALLETS ARE ON THE OUT SIDE

$I = I_{Tcr} + (I_T - I_{cr}) \left(\frac{M_{cr}}{M_S}\right)^3$

Ms	Ie / ITR	ITR / Ie	LOAD CASE
33.8	$\cancel{.90}$ .91	$\cancel{1.00}$ 1.11	1
50.4	$\cancel{.36}$ .44	2.24 $\cancel{2.78}$	2
62.9	$\cancel{.25}$ .35	2.87 $\cancel{4.00}$	3
79.4	$\cancel{.188}$ .297	3.37 $\cancel{5.32}$	4
91.4	$\cancel{.168}$ .28	3.58 $\cancel{5.95}$	5

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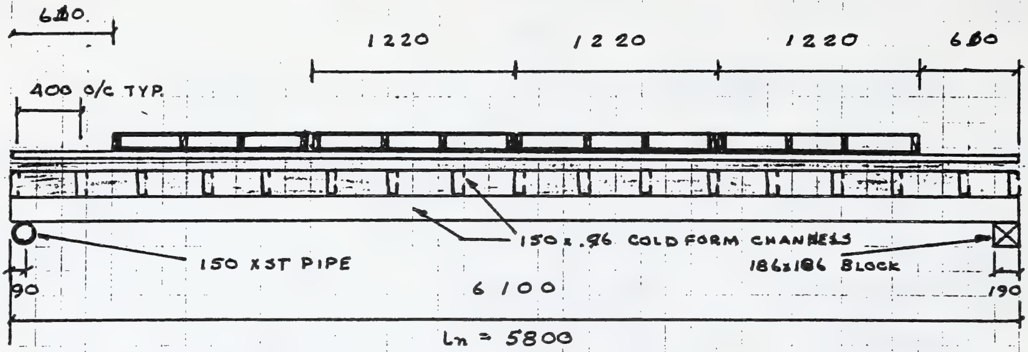
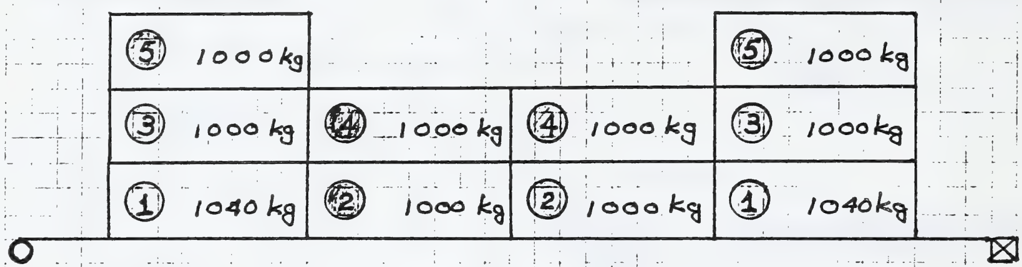
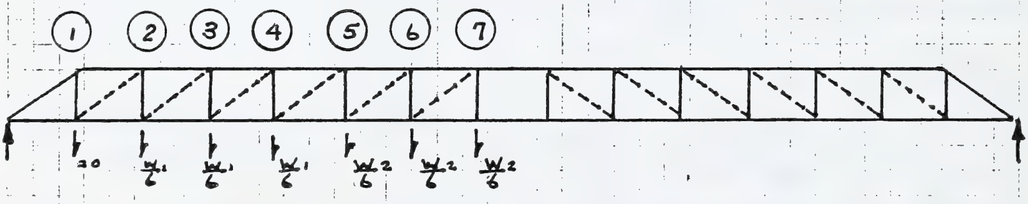
Beam Load Test



Load Sequence



Shear in Floor Joist



Load Case	1	2	3	4	5	6	7	M_{MID}	$M_{1/4}$	$\Delta_{1/2}$	$\Delta_{1/4}$
Ws	7.2	6.2	5.2	4.1	3.1	2.1	1.0	21.4	16.0	1.09	.81
1	5.1	5.1	3.4	1.7	0	0	0	12.4	11.8		
2	10.0	10.0	8.3	6.6	4.9	3.3	1.6	29.0	21.6	3.29	2.47
3	14.9	14.9	11.6	8.3	4.9	3.3	1.6	41.5	33.5		
4	19.8	19.8	16.5	13.2	9.9	6.5	3.3	58.0	43.5	10.55	7.33
5	24.3	24.3	19.8	15.4	10.9	6.5	3.3	70.0	54.9		
Wt	31.5	30.5	25.0	20.5	14.0	8.6	4.3	91.4	70.3		

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STRESS CALCULATIONS

1) $M = A_s f_s (d - \frac{a}{2})$ $\qquad a = \frac{A_s f_s}{b\, f_c/2}$

2) $M = A_s f_s (d - \frac{a}{2})$ $\qquad a = \frac{A_s f_s}{b \times .85 f_c}$

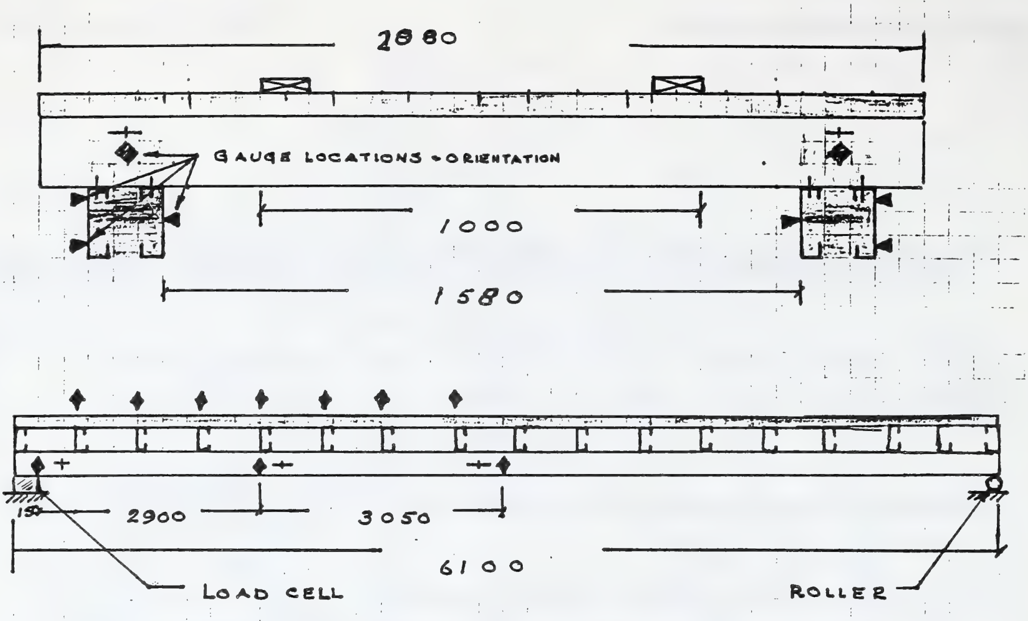
3) $M = f_s S_{bt}$ $\qquad y = 217$, $= \cancel{141}$ 141

4 $M = f_s S_{be}$ $\qquad s_{be} = I_e / y$ $\qquad y = \cancel{184}$ 327, $= \cancel{108}$ 251

LOAD CASE	$f_s = M/(d-\frac{a}{2})A_s$		$f_s = M/S_{bt}$		$f_s = M/S_{be}$	
	BOTTOM	TOP	y = 217	y = 141	y = 327	y = 251
1: 12.4	43	43	7.6	4.95	12.62	9.69
11.8	41	41	7.24	4.71	[illegible]	[illegible]
2: 29.0	101	101	17.80	11.57	[illegible]	[illegible]
21.6	75	75	13.26	8.61	[illegible]	34.25
3: 41.5	145	145	25.43	16.55	110.13	34.57
33.6	117	117	20.63	13.40	[illegible]	[illegible]
4: 58.0	205	205	35.60	23.13	180.80	133.73
43.5	152	152	26.70	17.35	135.61	[illegible]
5: 70.0	245	245	42.97	27.92	231.81	177.04
54.9	194	194	33.70	21.90	181.31	139.55

MOMENT CAPACITY

ALTERNATIVES	M TEST [kNm]	M PRED.	M TEST / M PRED
$f_y S_{bt}$	91.4	425.6	.215
$f_y S_{bce}$	91.4	73.0	1.25
$A_s f_y (d-\frac{a}{2})$	91.4	86.7	1.054
$A_s f_u (d-\frac{a}{2})$	91.4	98.1	.93



INSTRUMENTATION

LOAD CASE	1/4 POINT mm	MID SPAN mm	1/4 POINT mm	
INITIAL READING	0	0	0	DIAL
	1.25	2.5	3.75	SURVEY
1	1.57	4.24	1.47	DIAL
	4.75	4.75	6.25	SURVEY
2	8.94	11.8	9.02	DIAL
	11.25	11.5	13.75	SURVEY
3	11.18	15.18	28.27	DIAL
	14.5	18.0	16.5	SURVEY
4	13.96	43.1	37.76	DIAL
	24.25	29.5	23.75	SURVEY

CAMPBELL WOODALL & ASSOCIATES CONSULTING ENGINEERS LTD. JOB NO: 189 - 982-04
PROJECT: GEMINI FLOOR BEAM DATE:
CHECKED BY: DESIGN BY: D THOMPSON SHEET NO:

APPENDIX C

COLUMN TEST RESULTS AND ANALYSIS

Our Project No. CB00020
Your Reference No.

July 11, 1989

Campbell Woodall and Associates
Consulting Engineers Ltd.
250, 1210 - 8 Street S.W.
Calgary, Alberta
T2R 1L3

Attention: Mr. Dave Thompson, P. Eng.

Dear Sirs;

RE: Load Test on Columns

In response to your request, ultimate axial compressive strength of three columns was determined. The columns were 2.44 meters in height and had a 150 x 150 mm square section. Two of the columns consisted of four steel channels filled with concrete, while the third column consisted of two steel channels filled with concrete and reinforced with wire mesh.

The axial loading program was conducted in a steel frame with a hydraulic loading jack. In conjunction with the column load testing, compressive strength of the concrete was determined by testing two 100 x 200 mm cylinders.

The results of concrete compressive strength and column axial load tests are presented in Tables 1 to 4 respecitvely.

TABLE 1
Concrete 28-Day Compressive Strength

Cylinder	Compressive Strength (MPa)
1	26.5
2	26.1

Data in Tables 2 to 4 present load and column shortening due to the applied loads as well as lateral deflection caused by the loads applied. This lateral deflection was monitored on faces at 90 degrees to each other.

TABLE 2
Load Test on Columns No. 1

Load (KN)	Axial Deflection (mm)	Mid Height Horizontal Deflection (mm) Perpendicular Faces	
75.6 (17,000 lbs)	1.52	0.44	0.97
125.7 (28,250 lbs)	2.49	0.65	1.31
175.7 (39,500 lbs)	4.06	1.99	0.68
275.0 (61,833 lbs)	6.10	2.21	0.65
372.3 (83,667 lbs)	7.92	3.24	0.16
469.5 (105,500 lbs)	9.65	4.81	0
564.9 (127,000 lbs)	11.68	5.79	-3.55
653.9 (147,000 lbs)			

Ultimate Strength = 653.9 KN (147,000 lbs) *
* Failure was by concrete crushing at the lower secion of the column.

TABLE 3
Load Test on Columns No. 2

Load (KN)	Axial Deflection (mm)	Mid Height Horizontal Deflection (mm) Perpendicular Faces	
75.6	1.36	.55	0
125.7	2.33	1.27	0
175.7	3.67	1.73	0
275.0	5.37	2.79	-0.25
372.0	7.30	3.93	-0.66
469.5	8.90	4.88	-1.22
564.9	11.50	5.84	-1.91
653.9			

Ultimate strength = 564.51 KN (127,000 lbs) *
* Failure was by concrete crushing at the lower section of the column.

TABLE 4
Load Test on Columns No. 3
(with Two Steel Channels and Concrete Reinforced
With Wire Mesh)

Load (KN)	Axial Deflection (mm)	Mid Height Horizontal Deflection (mm) Perpendicular Faces	
75.6	1.37	1.12	0.51
125.7	2.57	1.83	0.88
175.7	3.43	2.44	0.91
275.0	5.08	3.18	0.66
372.0	6.91	3.60	0.53
469.5	8.71	3.81	0.15
564.9	10.41	3.68	-0.63
653.9	12.14	3.61	-1.91
689.4			

Ultimate Load = 689.4 KN (155,000 lbs) *
* Failure was by concrete crushing at the lower section of the columns.

We trust the above information meets your requirements. If you have any questions or require us to serve you further, please call us.

PERMIT TO PRACTICE
HARDY BBT LIMITED
Signature DE
Date 89.07.12
PERMIT NUMBER: P 4546
The Association of Professional Engineers, Geologists and Geophysicists of Alberta

Yours truly

Per:

V. Yogendran, ~~Ph.D.~~, E.I.T.

Per:

PROFESSIONAL ENGINEER
D. EMPEY
ALBERTA

D. Empey, P. Eng.
Manager, Materials Engineering
Division

VY:db

Hardy BBT Limited

CONSULTING ENGINEERING & PROFESSIONAL SERVICES

Our Project No.
Your Reference No. CB-00020

June 9, 1989

Campbell Woodall and Associates
Consulting Engineers Ltd.
250, 1210 - 8 Street S.W.
Calgary, Alberta
T2R 1L3

Attention: Mr. Dave Thompson, P. Eng.

Dear Sir:

RE: Push Out Strength and
Ultimate Strength of Column Sections

In response to your request, push out strengths of six column sections and ultimate strengths of 12 column sections were determined. The column sections were 6"x 6" square made of four steel channels filled with concrete.

The compressive strength of the concrete was determined by testing three 4" x 8" cylinders. The tensile strength of the steel was determined from two samples. The deformation of the samples were measured in most of the tests. The deformation was measured from the load of 200 lbs. to failure. The results obtained are attached.

We trust this information meets your requirements. If you have any questions please call us.

PERMIT TO PRACTICE
HARDY BBT LIMITED
Signature
Date 89.06.12
PERMIT NUMBER: P 4546
The Association of Professional Engineers, Geologists and Geophysicists of Alberta

Yours truly,

Hardy BBT Limited

Per:

V. Yogendran, Ph.D., E.I.T.

Per:

D. Empey, P.Eng.
Manager, Materials Engineering Division

VY:bb

219 - 18 STREET S.E. CALGARY, ALBERTA T2E 6J5 TELEPHONE (403) [illegible] FAX (403) 248-2188

COMPRESSIVE STRENGTH OF CONCRETE

Sample	Compressive Strength (MPa)
#1	23.5
#2	23.0
#3	23.0

Lab. Order No.	CB00020
Type of Sample	
Project	
Source	
Sampled by	
Date Sampled	
Date Received	
Date Tested	June 9, 1989
Date Reported	
Laboratory	

Copies to:

METAL TEST REPORT

:

. Tension Tests

Sample Mark	1	2
Size	.492 x .039 .390 x .025	.493 x .039 .374 x .026
Init. Area-sq. ins.	.0192	.0192
Final Area-sq. ins.	.0097	.0097
Total Load-lbs.	962	972
Ult. Stress-psi.	50, 100	50, 600
Yield Load-lbs.	850	840
Yield Stress-psi.	44, 300	43, 700
Init. Gage-ins.	2.000	2.000
Final Gage-ins.	2.680	2.715
Elongation-percent	34.0	35.7
Red. in Area-percent	49.5	49.5
Type of Failure	Ductile	Ductile
Fracture		

. Bend Tests

Sample Mark

Passed-Failed

. Other Tests

. Remarks

Certified: ....

HT 22 - 79/06

PUSH OUT TEST

SAMPLE #1

DEFORMATION (In. x 10 –3)	LOAD (Lbs.)
10	1100
20	2100
30	3600
40	6600
50	8400
60	10000
70	11500
80	12800
90	13600
110	14200
110	14600
120	14800
130	14800
140	14800
150	14850
160	14850

NOTE: Steel trimmed 2" from the concrete

PUSH OUT TEST
SAMPLE #2

DEFORMATION (In. x 10 -3)	LOAD (Lbs.)
10	1200
20	3800
30	6500
40	8800
50	11600
60	13200
70	13700
80	14200
90	14800
110	15300
110	15800
120	16000
130	16000
140	16100
150	15700
160	15600
170	15600
180	15700
190	15800
200	15500
210	15600
220	15600
250	15800
300	9600
350	7800

NOTE: Steel trimmed 2" from the concrete, not completely square.

PUSH OUT TEST
SAMPLE #1
RERUN

DEFORMATION (In. x 10 –3)		LOAD (Lbs.)
10		1300
20		4000
30		5800
40		7200
50		9750
60		11000
70		11800
80		12700
90		13600
100		14000
110		14700
120		14500
130		14700
140		14700
150		14800
160		14800
180	Movement Concrete	14200
190		13800
200		13600

NOTE: Steel trimmed 1/4" from the concrete.

PUSH OUT TEST

SAMPLE #2
RERUN

DEFORMATION (In. x 10 -3)		LOAD Lbs.
10		3100
20		9600
30		19800
40		33500
50		48000
60		53800
70		60200
80	Slight Movement	62600
90		66500
100		71200
110		72200
120		75800
130		79500
136		80000

NOTE: Steel trimmed almost flush with concrete.

PUSH OUT TEST
SAMPLE #3

DEFORMATION (In. x 10 -3)	LOAD (Lbs.)
10	1500
20	3800
30	7600
40	12200
50	15000
60	17000
70	18000
80	18500
90	19100
100	19900
110	20400
120	20500
130	20800
140	21100
150	21600
160	21800
170	21700
180	22200
190	23100
200	24400
210	25900
220	27400
230	28600
240	29800
250	30800
260	32800
270	35000
280	36600
290	38600
300	41500
310	45000
320	50000
330	59000
340	66000
350	74800
354	80000

NOTE: Steel trimmed 1/4" below the concrete.
Concrete flush with bottom of steel at removal.

PUSH OUT TEST
SAMPLE #4

DEFORMATION (In. x 10 -3)		LOAD (Lbs.)
10		2300
20		7000
30		11450
40		14000
50		15500
60		16600
70	Slight Movement Down	17000
80		17200
90		17300
100	Movement Down 1/8"	17300
110		17300
120		16900
130		16300
140		16000
150		16000
200		16300
250		18100

Steady Load Climb.

NOTE: Steel trimmed 1/4" below concrete.

PUSH OUT TEST
SAMPLE #5

DEFORMATION (In. x 10 –3)		LOAD (Lbs.)
10		2500
20		5900
30		10400
40		13500
50		15300
60		17000
70		18000
80		18800
90		19500
100		20100
110		20900
120		22000
130		23100
140		24400
150		26400
160	Movement	28100
170		30300
180		33500
190		38000
200		41100
250		62400

NOTE: Steel trimmed 1/4" below the concrete.

PUSH OUT TEST
SAMPLE #6

DEFORMATION (In. x 10 –3)		LOAD (Lbs.)
10		2100
20		6200
30		10500
40		13100
50		14600
60		15700
70		16300
80		16500
90		16400
100		16500
110		16600
120	1/8" Movement	16600
130		16700
140		17000
150		17100
160		17600
170		18000
180		18400
190		19000
200		19800
250		27600

NOTE: Steel was trimmed 1/4" below concrete.

ULTIMATE STRENGTH TEST

SAMPLE #1A

DEFORMATION (In. x 10 -3)	LOAD (Lbs.)
10	5000
20	9000
30	17500
40	20800
50	22200
60	54000
70	74600
80	85000
90	88500
100	85000
110	
120	74500
130	73000
140	55000

ULTIMATE STRENGTH TEST

SAMPLE #2A

Ultimate Strength – 120,000 lbs.

NOTE: Bottom End Capped

ULTIMATE STRENGTH TEST

SAMPLE #3A

DEFORMATION (In. x 10 -3)	LOAD (Lbs.)
10	6000
20	12000
30	23000
40	34000
50	54000
60	86000
70	110000
80	124000
90	125500

NOTE: Bottom End Capped.

ULTIMATE STRENGTH TEST

SAMPLE #4A

DEFORMATION (In. x 10 –3)	LOAD (Lbs.)
10	5500
20	12000
30	22500
40	33000
50	53000
60	71000
70	90000
80	101000
90	107000
93	109000

NOTE: Bottom End Capped.

ULTIMATE STRENGTH TEST

SAMPLE #5A

Ultimate Strength – 108,000 Lbs.

NOTE: Bottom End Capped.

ULTIMATE STRENGTH TEST

SAMPLE #6A

DEFORMATION (In. x 10 –3)	LOAD (Lbs.)
10	6000
20	15000
30	25000
40	40000
50	80000
60	90000
70 Failure	110000

NOTE: Bottom End Capped.

ULTIMATE STRENGTH TEST

SAMPLE #1B

Ultimate Strength – 118,000 Lbs.

NOTE: Bottom End Capped.

ULTIMATE STRENGTH TEST

SAMPLE #2B

DEFORMATION (In. x 10 -3)	LOAD (Lbs.)
10	6500
20	14000
30	31500
40	63000
50	90000
60	102500
70	113000

ULTIMATE STRENGTH TEST

SAMPLE #38

DEFORMATION (In. x 10 –3)		LOAD (Lbs.)
10		6000
20		13000
30		25000
40		41000
50		75000
60		100000
70		118000
75	Failure	120000

ULTIMATE STRENGTH TEST

SAMPLE #4B

DEFORMATION (In. x 10 –3)		LOAD (Lbs.)
10		6500
20		14000
30		27000
40		51000
50		82500
60		99500
70		110000
80	Failure	115000

ULTIMATE STRENGTH TEST

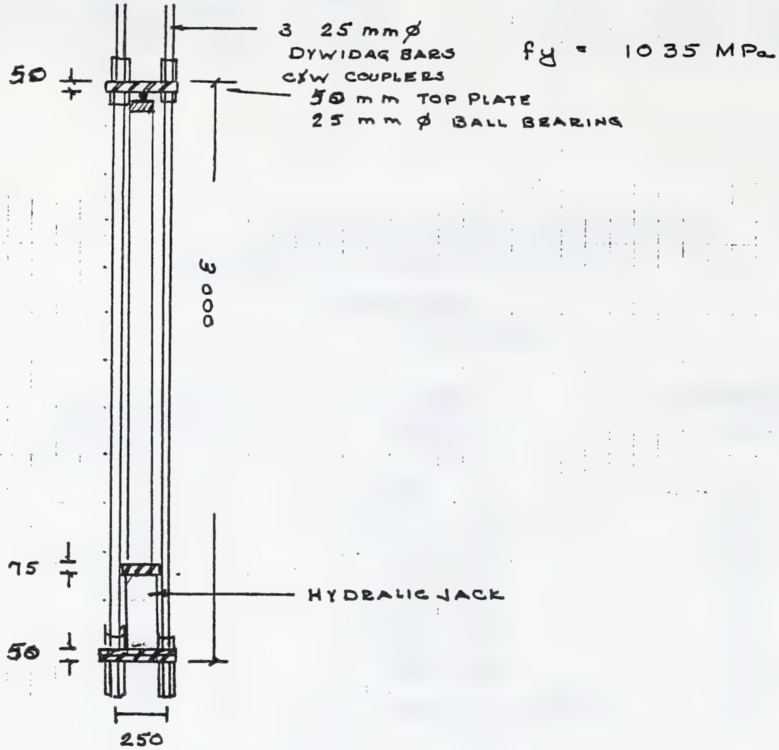
SAMPLE #5B

DEFORMATION (In. x 10 -3)		LOAD (Lbs.)
0		6500
20		14000
30		25500
40		49000
50		81000
60		100500
70		114000
75	Failure	115500

ULTIMATE STRENGTH TEST

SAMPLE #68

DEFORMATION (In. x 10 -3)		LOAD (Lbs.)
10		6500
20		14500
30		27000
40		51000
50		81000
60		98500
70		110000
80	Failure	114000



THE DYWIDAG BARS WILL ELONGATE DURING LOAD TEST

USING $E = 200$ GPa

$A_s = 3 \times 548 \text{ mm}^2 = 1644$ mm

$$\Delta = \frac{PL}{AE} = \frac{P \times 3000}{1644 \times 200} \qquad P = [\text{kN}]$$

$$= .0091 \text{ mm/kN} \qquad \frac{P_{max}}{A} = 419 \text{ MPa} < f_y$$

CHANGE IN LENGTH DUE TO ELONGATION OF DYWIDAGE BARS

$$\varepsilon = \frac{\Delta - \Delta'}{2440}$$

LOAD [kN]	Δ' [mm]
75.6	.69
125.7	1.15
175.7	1.60
275.0	2.51
372.0	3.39
469.5	4.28
564.9	5.15
653.9	5.97

CAMPBELL WOODALL & ASSOCIATES CONSULTING ENGINEERS LTD.	JOB NO: 189-984-02
PROJECT: Gemini Column	DATE:
CHECKED BY: DESIGN BY: D Thompson	SHEET NO:

COLUMN #1

LOAD [kN]	Δ [mm]	Δ'	$\frac{\Delta - \Delta'}{2440}$	$\frac{P}{A_T E}$	$\frac{\Delta_{TEST}}{\Delta_P}$	Δ_{MID}	Δ_P	$\frac{\Delta_{TEST}}{\Delta_P}$
75.6	1.52	.69	.0003	.0001	4.09	1.07	-	-
125.7	2.49	1.15	.0006	.0001	3.98	1.46	1.56	1.93
175.7	4.06	1.60	.0012	.0002	6.35	2.10	1.62	1.30
275.0	6.10	2.51	.0015	.0003	4.86	2.30	1.74	1.32
372.3	7.92	3.39	.0019	.0004	4.52	3.24	1.33	1.73
469.5	9.65	4.28	.0022	.0005	4.25	4.81	2.03	2.37
564.9	11.68	5.15	.0027	.0006	4.30	6.79	2.21	3.07
653.9	–	–	–					

COLUMN 2

LOAD	Δ	Δ'	$\frac{\Delta - \Delta'}{L}$	Δ_P	$\frac{\Delta_{TEST}}{\Delta_P}$	Δ_{MID}	Δ_P	$\frac{\Delta_{TEST}}{\Delta_P}$
75.6	1.36	.69	.0003	.0001	3.3	.55	–	–
125.7	2.33	1.15	.0005	.0001	3.5	1.27		
175.7	3.67	1.60	.0008	.0002	4.38	1.73		
275.0	5.37	2.51	.0012	.0003	3.87	2.80		
372.0	7.30	3.39	.0016	.0004	3.91	3.99		
469.5	8.90	4.28	.0019	.0005	3.66	5.03		
564.9	11.50	5.15	.0026	.0006	4.18	6.14		
564.5	–	–	–			–		

$$\Delta_T = \frac{P}{A_T E_c}$$

$$E_c = 5 \times \sqrt{26.3} = 25.64 \text{ GPa}$$

$$A_T = \frac{.039}{.036} \times 2 \times (230 + 200) \times \left[\frac{195}{5\sqrt{26.3}} - 1\right] + 152 \times 152 = 35.41 \times 10^3 \text{ mm}^2$$

$$A_c = 22.1.7 \times 10^3 \text{ mm}^3$$

CAMPBELL WOODALL & ASSOCIATES CONSULTING ENGINEERS LTD.		JOB NO: 189-984-02
PROJECT: GEMINI COLUMN		DATE:
CHECKED BY:	DESIGN BY: D. THOMPSON	SHEET NO:

COLUMN #3

LOAD	Δ	Δ'	$\frac{\Delta - \Delta'}{L}$	Δ p	$\frac{\Delta_{TEST}}{\Delta_P}$	Δ MID	Δ P	$\frac{\Delta_{TEST}}{\Delta_P}$
75.6	1.37	.69	.0003	.0001	3.08	1.23	–	–
125.7	2.57	1.15	.0006	.0002	3.87	2.03	1.[illegible]	1.13
175.7	3.43	1.60	.0007	.0002	3.56	2.60	1.86	1.4[illegible]
275.0	5.08	2.51	.0011	.0003	3.20	3.25	2.00	1.62
372.0	6.91	3.39	.0014	.0004	3.23	3.64	2.16	1.69
469.5	8.71	4.28	.0018	.0006	3.22	3.81	2.33	1.63
564.9	10.41	5.15	.0022	.0007	3.18	3.73	2.54	1.4[illegible]
653.9	12.14	5.97	.0025	.0008	3.23	4.08	2.75	1.43
689.4								

CAMPBELL WOODALL & ASSOCIATES CONSULTING ENGINEERS LTD. JOB NO: 189-984-02

PROJECT: GEMINI COLUMN DATE:

CHECKED BY: DESIGN BY: D THOMPSON SHEET NO:

COLUMN TEST SUMMARY

TEST	P TEST [KIPS]	.85Acfc	P TEST / Pp	fyAs + .85Acfc	P TEST / Pp	.85 Ac fcc2	
SHORT COLUMN "A"	114.5	102.9	1.11	118.7 *	.97	108.7	1.057
SHORT COLUMN B	115.9	102.9	1.12	118.7 *	.98	102.9	1.12
LONG COLUMN 1	147	116.7	1.25	145.9	1.007	122.4	1.20
LONG COLUMN 2	127	116.7	1.09	145.9	.87	122.4	1.038
LONG COLUMN 3	155	116.7	1.32	153.9 +	1.007	122.4	1.27

* $A_s f_y = 17.88^k$ BASED ON PUSH OUT TESTS

$A_s f_y = 2 \times .358 \times 44 = 31.5$

+ $A_s f_y = 2 \times .358 \times 44 + 4 \times .04 \times 50 = 39.5^{k}$

$f_{cc2} = f'_c + 2C f_y t / D$ D = 1.6 t .036

C = 1.9 fs = 12.7 ksi

CONCLUSION

1.) COMPOSITE ACTION CAN BE OBTAIN WITH STRAIGHT CHANNEL

2.) SCREWED SECTION HAS MORE DUCTILE BEHAVIOUR

3.) USING $f_y A_s + .85 A_c f_c$ GIVE BEST RESULTS

APPENDIX D

PUSHOUT TESTS

OF

SHEAR TABS

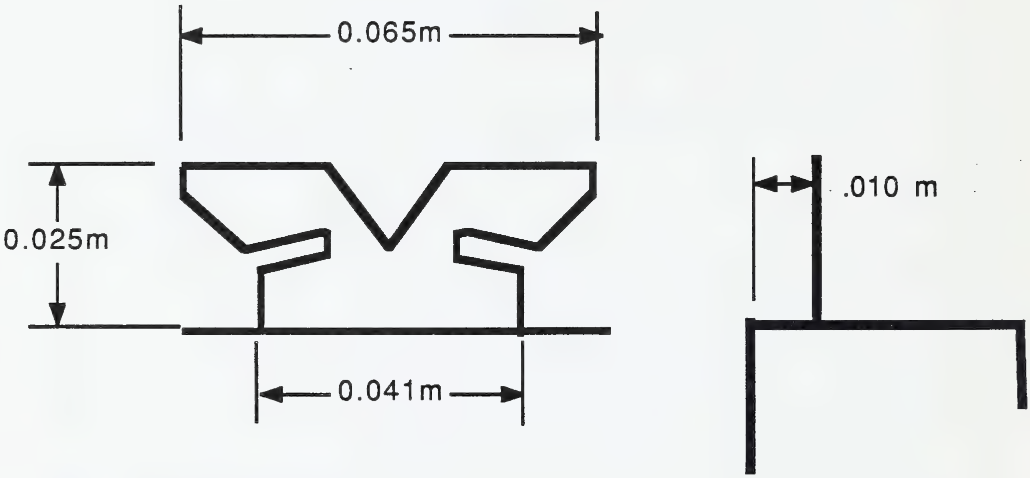
FOR

GEMINI FLOOR AND WALL SYSTEMS

David P. Thompson, P. Eng., MSc.
Campbell Woodall & Associates
Consulting Engineering Ltd.

1.0 Introduction

Composite system behaviour is largely dependant on the interconnection between the two different materials. The objective of this portion of the overall experimental program was to identify the capacity of these shear tabs (see Figure D.1) to transfer load between the cold-formed steel channels and the concrete topping.



SHEAR TAB
FIGURE D.1

2.0 Test Program

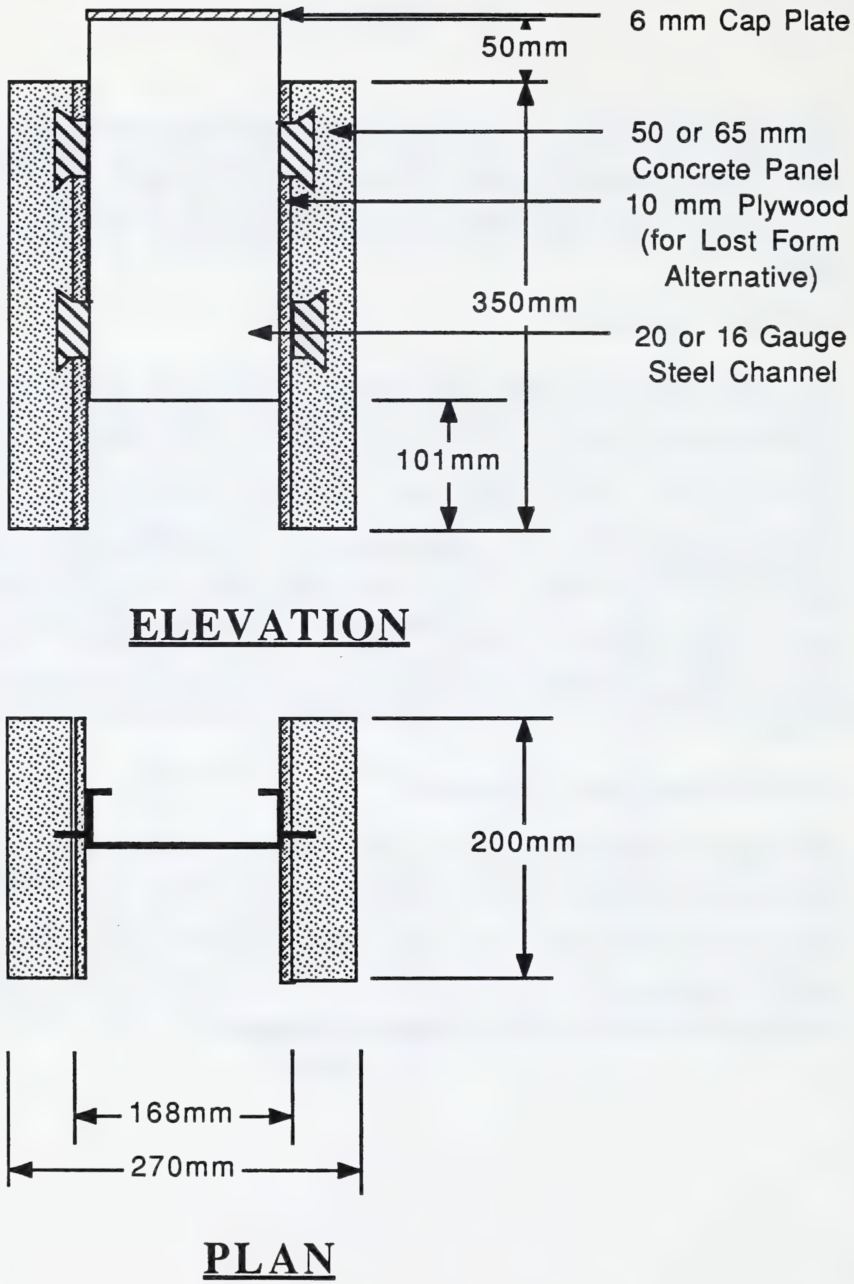
The test method used was to axially load a steel cold-formed channel which was attached to two concrete panels. This arrangement permitted the shear tabs to be symmetrically loaded. This type of test has been used for finding the capacities of Nelson Studs and is considered to simulate the loading conditions on the shear connection (tabs) that occur in the composite system.

2.1 Test Specimens

The test specimens were fabricated according to the drawings in Figure D.2. Four shear tabs were embedded in the two, 350 mm x 200 mm concrete panels, permitting symmetric loading on the channel. The shear tabs in the channels were spaced at 150 mm which is the same as for the new floor system. The channels were either fully embedded in the concrete, simulating the reusable form floor system and precast wall panels; while 10 mm plywood was used to separate the concrete from the channel simulating the lost form floor system or walls if thermal breaks in the wall are required. The top 6 mm plate was used to allow even distribution of the load and to reduce the possibility of buckling of the unsupported channel. In the last series of tests (20 gauge channels at 21.6 and 26.1 MPa) a metal plate was used to reinforce the channel web to prevent buckling instead of the 6 mm top plate.

2.2 Test Setup and Procedure

The tests were carried out on a C.S.A. approved MTS machine used by Hardy BBT for testing concrete cylinders. The loads were measured from the machine while the slip was measured using a dial gauge. The dial gauge was attached to one of the concrete panels and measured the movement of the head of the machine. The dial gauge was capable of reading deflections of a $1/1000^{th}$ inch. The test was stopped when unrestrained deflection of the channel was observed.



PUSHOUT TEST SPECIMEN
FIGURE D.2

3.0 Observations and Evaluation of Results

3.1 Observations

During the tests, two distinct modes of concrete failure occurred for the two types of composite systems. The failure mode in the lost form system was instigated by a pull out failure of the back of the shear tab. In all the test specimens observed, after failure a shear cone was found. In addition, the shear tab was badly deformed with ripping of the steel at the junction of the shear tab and the top flange of the channel. The load deflection curves plus Tables D.1 and D.2 indicate that the load capacity of the shear tabs, in the lost form system, is 60 to 70% of that in the reusable form system. The load deflection curves for the lost form system, Figures D.3 to D.5, are on page D-8. During the the test it was observed that the downward deflection recorded occurred with the tab rotating between the concrete shell and the top of the channel flange until hinges were formed both at the channel and concrete. The shear capacity was maintained after the hinges were formed and failure was defined as unrestrained deflection.

The concrete panels sheared during the testing of the reusable form system. There was no deformation of the steel cold-formed tab except at the base of the tab, and the concrete was sheared right through. The plane of failure was at the shear tab; therefore, we concluded that the concrete that went through the hole in the top flange did not contribute to the shear capacity of the system. The load deflection graphs for the reusable form system are shown in Figures D.6 to D.8.

3.2 Evaluation of Results

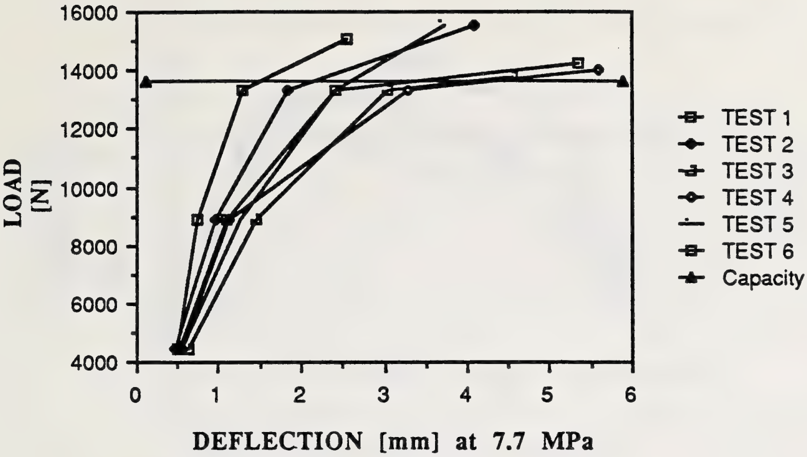
The results of the test program are summarized in Tables D.1 and D.2. The modified shear capacity of the shear tabs was calculated using ninety percent of the ultimate capacity to limit shear slip and then dividing the experimental results by the recorded yield strength and the minimum yield strength to be used for the gauge of metal in design (i.e. 230 MPa for 20 gauge and 345 MPa. for 16 gauge). No correlation was found between the concrete strength and the shear tab capacities but are relatively constant when the concrete strength is above 20 MPa. When the differences in yield strength and metal thicknesses were considered the shear tab capacities were within eight percent of each other. Tables D.1 and D.2 are the capacities of the test specimens (which have four shear tabs) and the modified capacities should be divided by four to have individual tab capacities. Using a "t" distribution and the experimental results the confidence level against using a Ø=.80 was 96 to 99 %.
Also 65 mm and 50 mm concrete panels were compared with the 16 gauge channels and no change in the shear tab capacities were recorded. This was also true when 150 x 150 wire mesh was put into the concrete panels, again no increase in strength was observed. The load deflection curves of these two last sets of tests are shown in Figures D.9 and D.10.

ULTIMATE CAPACITIES OF SHEAR TABS IN LOST FORM SYSTEM
TABLE D.1

STEEL GAUGE	F_y [MPa]	F_c [MPa]	Average Capacity [kN]	Standard Deviation [kN]	Modified Capacity [kN]
20	307.8	7.7	14.74	.797	9.90
20	290.2	21.6	15.05	1.43	10.75
20	290.2	26.1	15.20	1.67	10.85
16	355	25.4	29.88	1.23	26.15
16	355	31.2	31.47	1.22	27.50

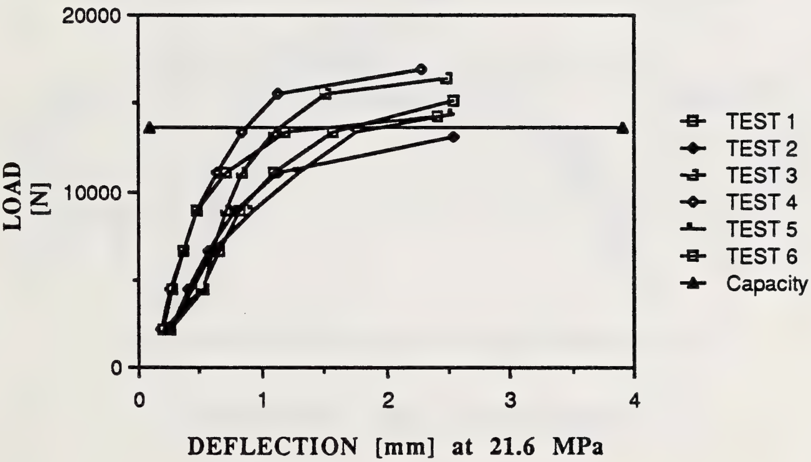
ULTIMATE CAPACITIES OF SHEAR TABS IN REUSABLE FORM SYSTEM
TABLE D.2

STEEL GAUGE	F_y [MPa]	F_c [MPa]	Average Capacity [kN]	Standard Deviation [kN]	Modified Capacity [kN]
20	307.8	7.7	20.91	1.31	14.05
20	290.2	21.6	24.76	1.61	17.65
20	290.2	26.1	26.24	2.31	18.70
16	355	25.4	47.37	4.15	41.40
16	355	31.2	46.85	3.12	41.00



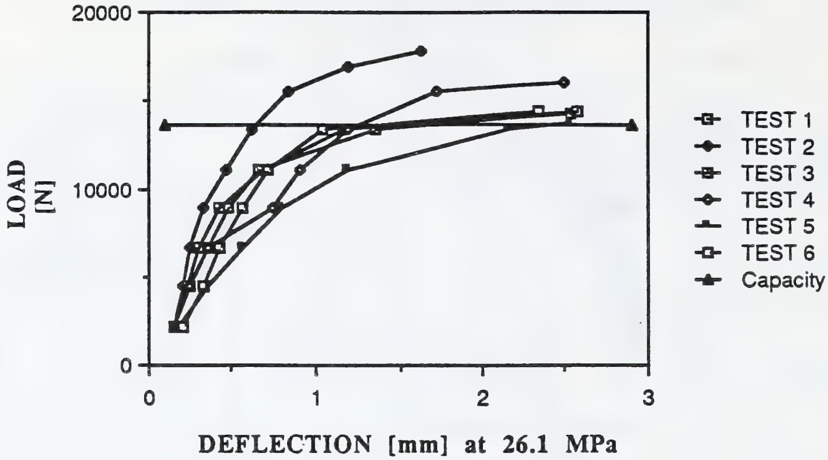
LOAD DEFLECTION CURVES FOR 20 GAUGE STEEL
IN LOST FORM SYSTEM

Figure D.3



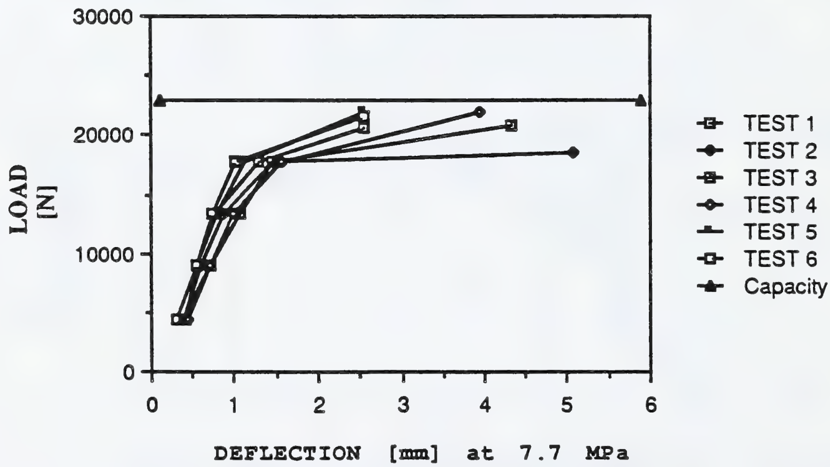
LOAD DEFLECTION CURVES FOR 20 GAUGE STEEL
IN LOST FORM SYSTEM

Figure D.4



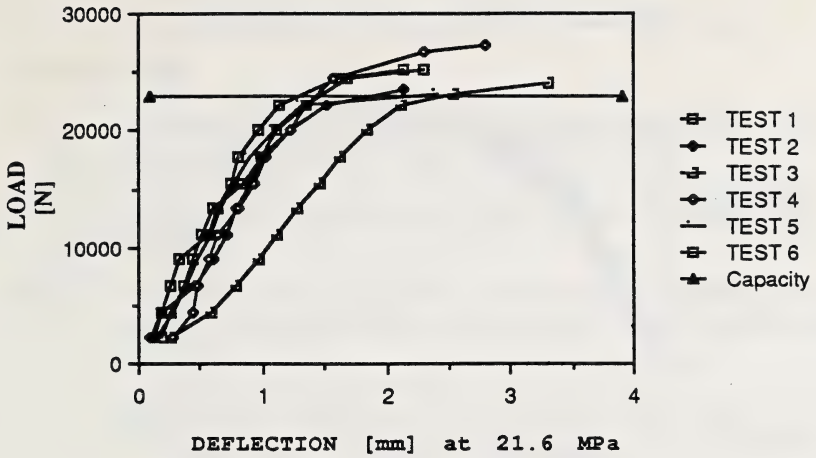
LOAD DEFLECTION CURVES FOR 20 GAUGE STEEL IN LOST FORM SYSTEM

Figure D.5



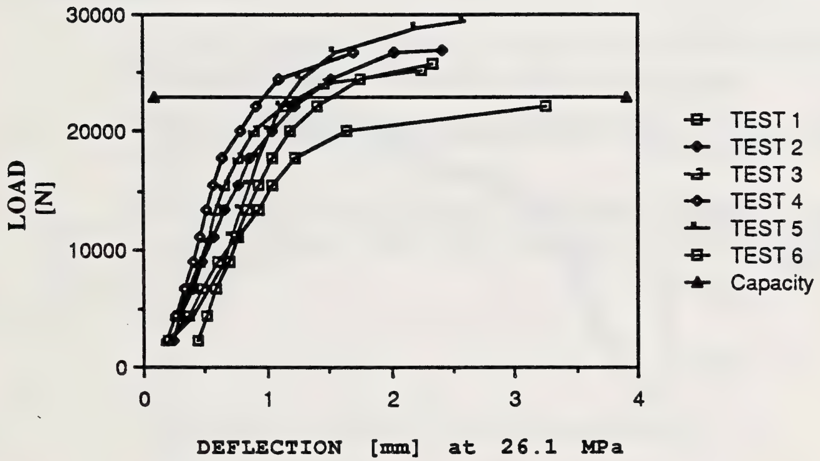
LOAD DEFLECTION CURVES FOR 20 GAUGE STEEL IN REUSABLE SYSTEM

Figure D.6



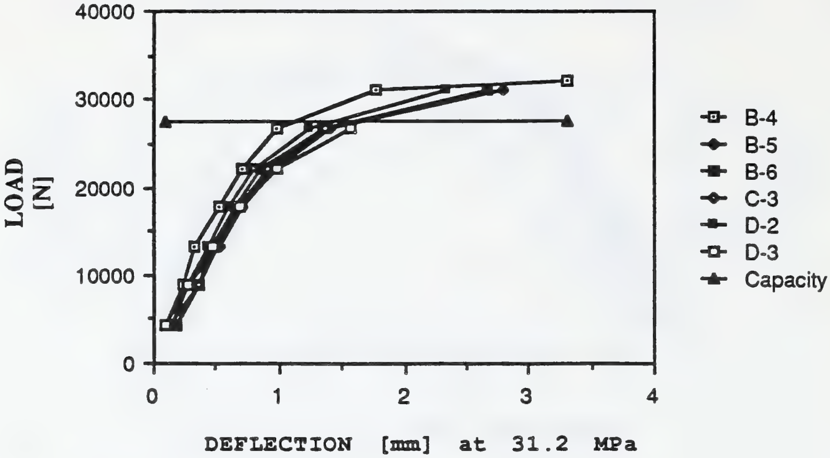
LOAD DEFLECTION CURVES FOR 20 GAUGE STEEL
IN REUSABLE SYSTEM

Figure D.7



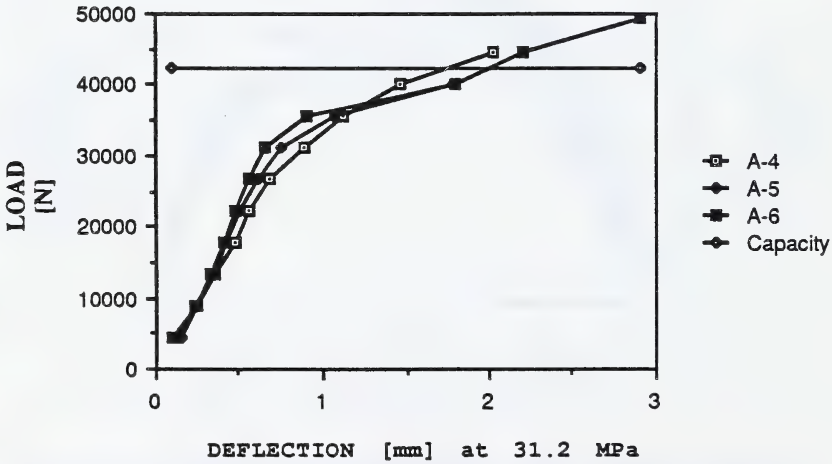
LOAD DEFLECTION CURVES FOR 20 GAUGE STEEL
IN REUSABLE SYSTEM

Figure D.8



LOAD DEFLECTION CURVES FOR 16 GAUGE STEEL
IN LOST FORM SYSTEM

Figure D.9



LOAD DEFLECTION CURVES FOR 16 GAUGE STEEL
IN REUSABLE FORM SYSTEM

Figure D.10

4.0 Conclusions and Recommendations

4.1 Conclusions

1 The thickness of the concrete topping or shell does not effect the capacity of the shear tab.

2 Reinforcing in the topping will not improve the shear tab's capacity.

3. In the reusable form system, the concrete through the hole where the shear tab was does not contribute to the shear tab capacity.

4 The shear tab capacity with concrete strength greater than 25 MPa is governed the the steel's strength and thickness.

5 The ultimate capacity of 4 shear tabs are given in Tables D.1 and D.2 for the two types of composite systems.

4.2 Recommendations

1 The values shown in Table D.3 be use with the joists supplied by Bailey Mantane in Canada.

2. A Ø factor of .80 should be used with the shear tab capacities in Table D.3.

3. For steel thicker than 16 gauge, the values for 16 gauge should be used for design unless further testing is done to confirm the higher values.

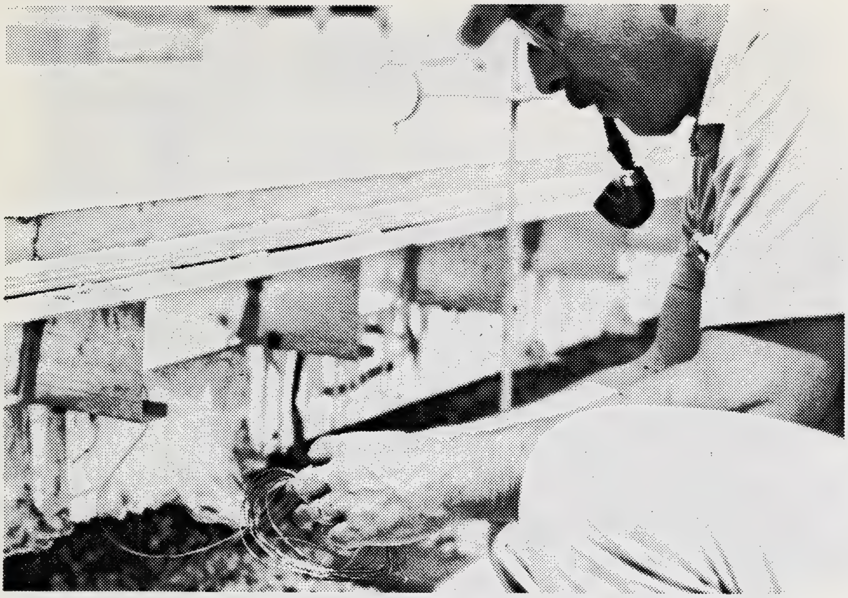
SHEAR TAB CAPACITY IN 25 MPa CONCRETE

TABLE D.3

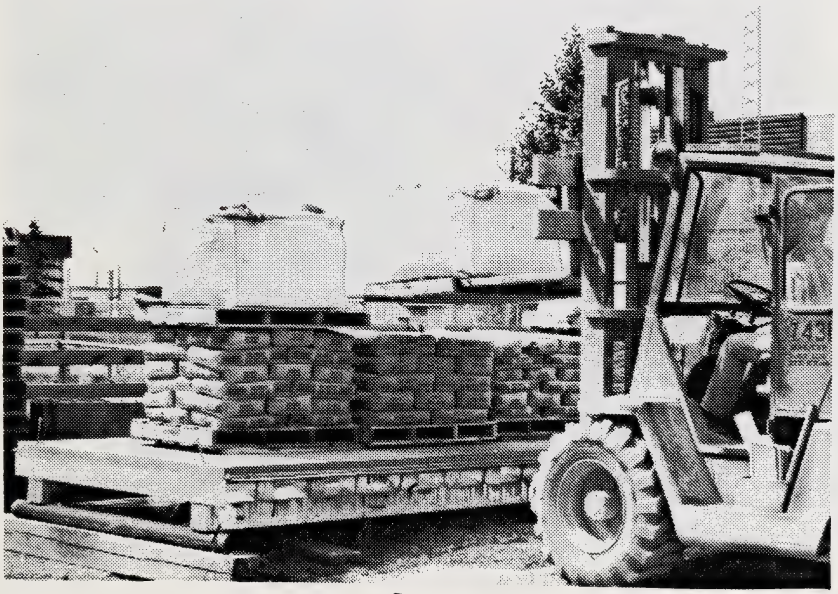
STEEL THICKNESS [GAUGE] [Thickness]	LOST FORM SYSTEM [kN]	REUSABLE FORM SYSTEM [kN]
20 [0.91 mm]	2.70	4.55
18 [1.21 mm]	3.50	5.50
16 [1.52 mm]	6.70	10.30

A P P E N D I X   E

P H O T O G R A P H S

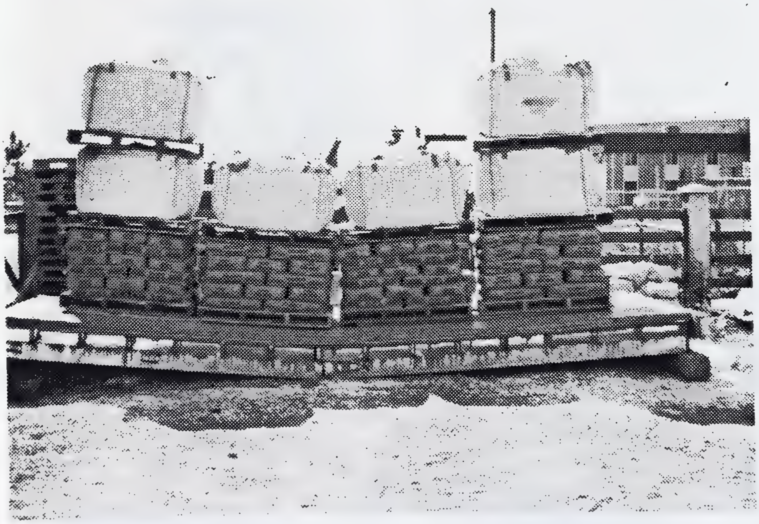


1. Installing instrumentation for full scale beam/panel testing.



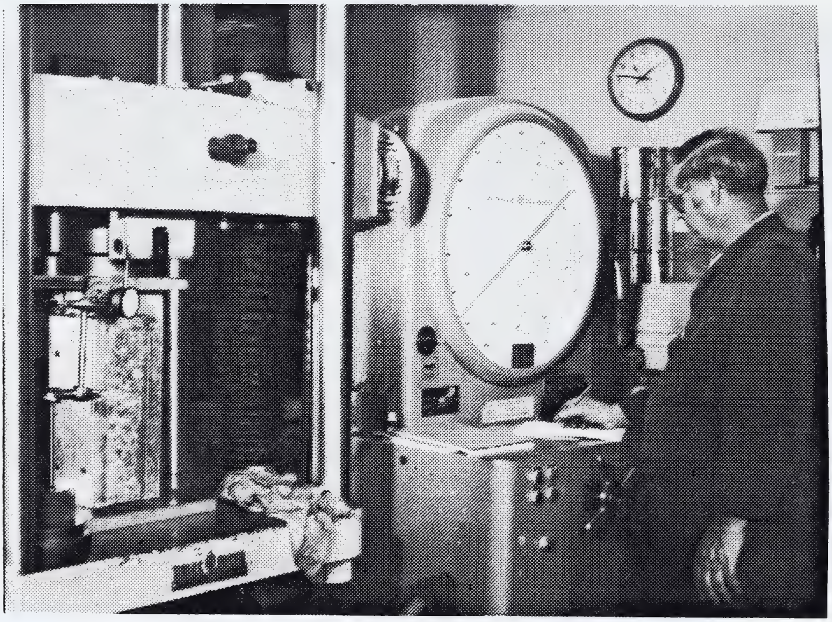
2. Application of load - full scale beam/panel testing.

3. Partially load beam/panel assembly.



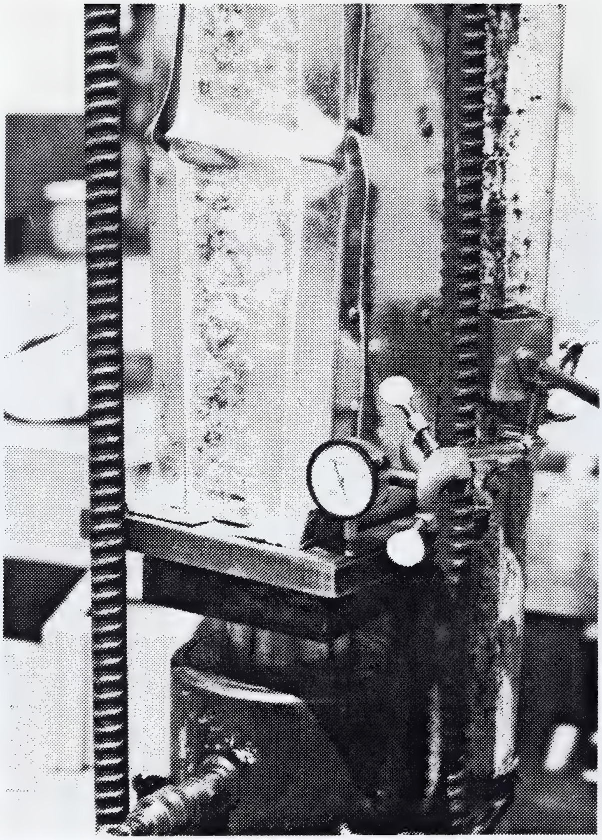
4. Beam/Panel assembly at failure.

5. Close-up showing tensile failure of cold-formed steel beam channels in beam/panel assembly.



6. Test setup for load application on short column sections.

7. Test setup for load application, full scale column testing.



8. Buckling of cold-formed steel channels at failure, full scale column testing.

9. Crushing of concrete at failure, full scale column testing.